AN INVESTIGATION OF CERTAIN STRENGTH-MOISTURE RELATIONSHIPS OF PARTIALLY SATURATED, UNDISTURBED COHESIVE SOILS

A THESIS

SUBMITTED TO THE FACULTY OF THE

GRADUATE SCHOOL OF THE

UNIVERSITY OF ALBERTA

BY

ANDREW LABARRE

IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

APRIL

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CONTENTS

PART I

RESULTS OF PREVIOUS RESEARCH PROGRAMS ON TRIAXIAL COMPRESSION TESTS OF SOILS

1. INTRODUCTION

From February 25 to 27, 1946, the Office, Chief of Engineers, U.S. Army, sponsored a meeting at which two reports[1] were given on certain investigations being carried out at that time in the field of soil mechanics. With regard to Parts I and II of this thesis the report given by Philip C. Rutledge, consultant of Northwestern University may be considered as the starting point. Both reports appeared in a publication released by the United States War Department in April 1947. Rutledge's findings are given in the following summary and they are used as the basis of the treatment of the triaxial and consolidation test data obtained by the University of Alberta on a research program for the Department of Transport, Canada, in Part II.

It is unfortunate that the D.O.T. tests were performed during 1946 and 1947 just prior to the release of the Rutledge report. Had this information been available sooner test procedures would have been somewhat altered. They are discussed in detail later.

2. CLASSIFICATION OF TRIAXIAL TEST RESULTS

Rutledge points out that the triaxial test data falls into three catagories. Each catagory depends on the particular type of soil and on the degree of saturation as follows:

(a) Cohesionless soils--e.g. gravels, sands, silty sands, rock flours, etc., both saturated and unsaturated.

(b) Saturated homogeneous soils--e.g. sedimentary clays below the zone of weathering and some undisturbed clays with as much as 5% of the voids filled with air.

(c) Partially saturated cohesive soils which includes the majority of the cohesive soils.

(1) The second report was prepared by Donald W. Taylor of the Massachusetts Institute of Technology and dealt with "Pressure Distribution Theories, Earth Cell Investigations, and Pressure Distribution Data".

e.g. re-compacted[1] saturated cohesive soils and natural cohesive soils, heterogeneous in structure.

3. BASIC TYPES OF TRIAXIAL TESTS

Important distinctions depending on the type of drainage occurring during the triaxial compression tests are made. Casagrande[2] points this difference out and classifies the tests under three groups:

(a) Quick tests, (Q), in which no drainage of pore water is permitted during the test.

(b) Consolidated Quick, (Q_c), in which drainage of pore water is permitted during the preliminary consolidation, namely, after applying the lateral pressure but before axial load increments are applied.

(c) Slow tests, (S), in which complete drainage of pore water takes place during all phases of the test.

Only (Q) tests were performed for the Department of Transport.

4. PRESENTATION OF TEST RESULTS

Plots of deviator[3] strength versus water content at the end of test have been shown to give the most direct and useful method of presenting the data test results. Fig. 1 reproduced from the Rutledge report for soft undisturbed Chicago Clay shows such a curve. Also illustrated is the major principal stress versus moisture content for the consolidation test assuming 100% saturation which Casagrande points out is another unique curve for a particular soil. For saturated cohesive soils it follows that the major principal stress, for any particular lateral pressure, versus moisture content at end of test is also a unique curve.

In the report are also shown similar curves for undisturbed Massena Clay. These, however, are not reproduced here. These distinctive curves lead Casagrande to formulate his working hypothesis.

(1) An extensive study was made on recompacted cohesive soils by R.H. Hislop in a thesis entitled, "Deformation Properties of Compacted Clay Soils", for a Masters Degree, University of Alberta, 1947.

(2) Professor A. Casagrande of Harvard University in the Harvard Report No. 3.

(3) Also called compressive strength and equals the diameter of the Mohr's Circle at failure conditions. e.g. $\sigma_1 - \sigma_3$

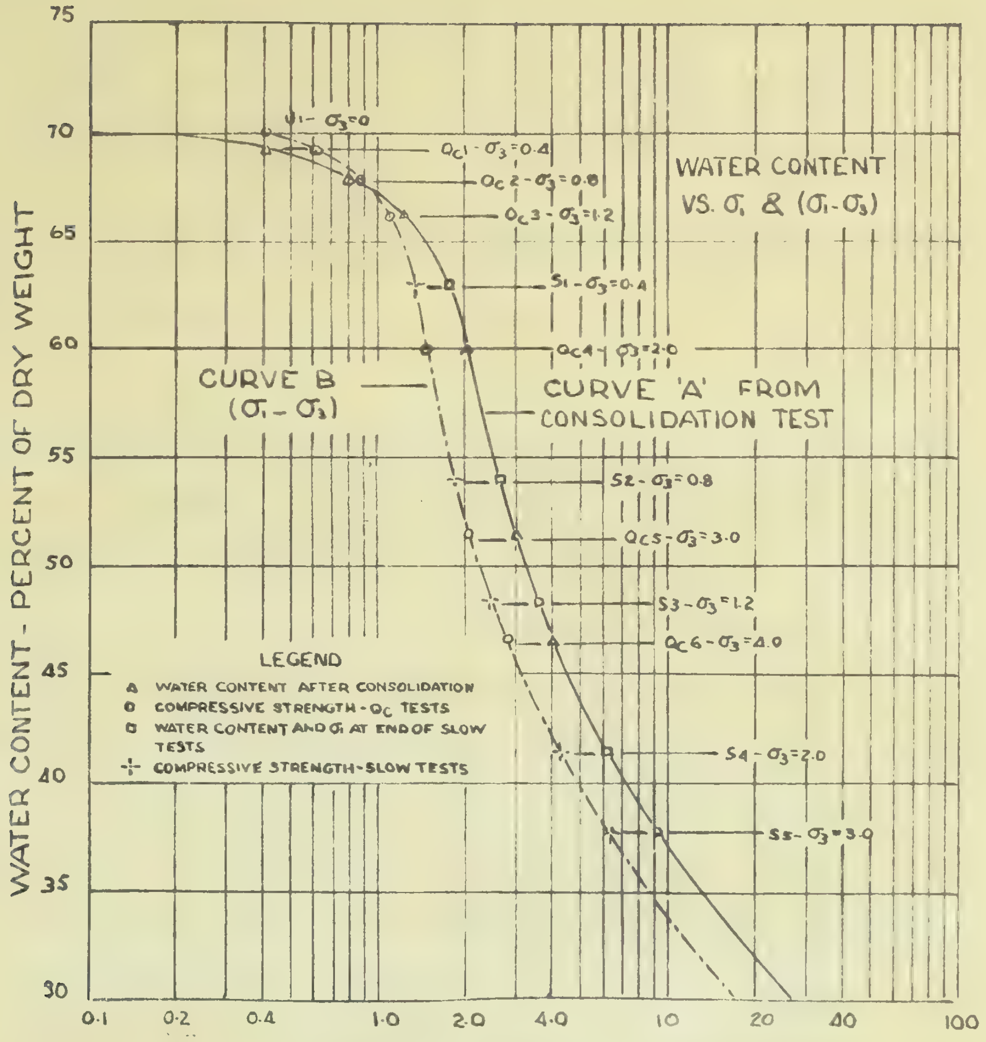


FIG 1 A

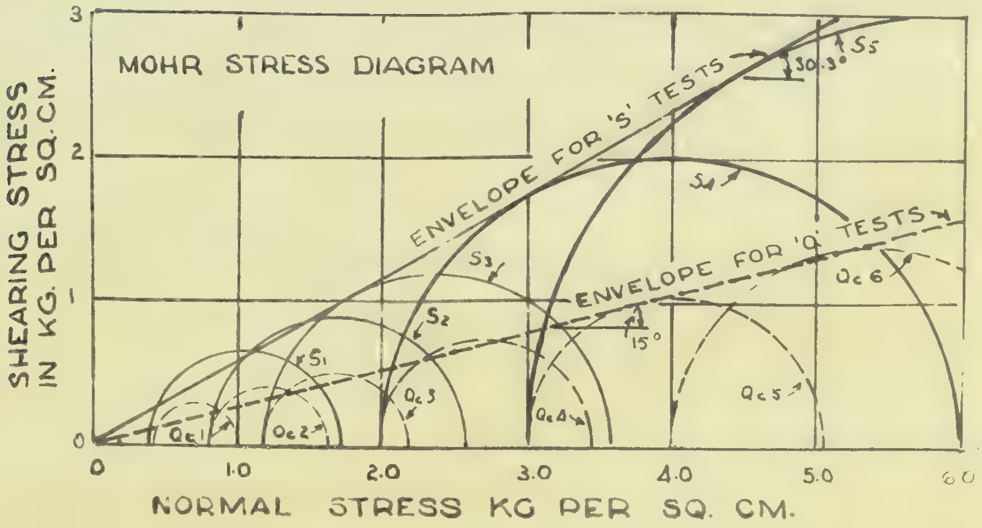


FIG 1 B

Fig. 1-A -- Water Content versus Major Principal Stress, Consolidation Test and Water Content versus Compressive Strength for Triaxial Test.

Fig. 1-B -- Mohr Stress Diagram for Failure Conditions Determined from Curves 'A' and 'B' in Fig. 1-A for 'Q_c' and 'S' Tests.

The above diagrams have been reproduced from the Rutledge Report.

Moisture contents instead of void ratio were selected for the compressive strength plots since this information was accurately available for the triaxial tests and could be readily computed for the consolidation tests. In many cases moisture contents appear to be the governing factor.

5. CASAGRANDE'S WORKING HYPOTHESIS FOR SATURATED COHESIVE SOILS

"For a saturated homogeneous clay the test results indicate that there is a single curve of compressive strength versus water content at end of test that is independent of the type of test, test lateral pressure and pore water pressure. The test data indicate a probability that there is a single curve of major principal stress, or consolidation, versus consolidated water content that is independent of the type of test and the minor principal stress".

The Rutledge Report offers considerable data to substantiate the working hypothesis using clays from Chicago, Massena, Atlantic City, Blue Mountain Dam and Boston. The information was provided by the various organizations that contributed to the U.S. War Department's[1] research program.

The two curves present a complete picture of the strength properties of saturated cohesive soils under the three types of drainage used in triaxial testing. From these curves the Mohr Circles and Mohr Rupture Line may be constructed. (See Fig. 1, Page 3)

6. SHEARING STRENGTH DETERMINATION USING CURVES 'A' AND 'B'

If the consolidation test curve is called 'curve A' and the deviator stress $\sigma_1 - \sigma_3$ versus % moisture at failure 'curve B', it can be shown that they enable the determination of the shearing strength in all three types of triaxial test. The procedures are:

A. For Quick 'Q' and Unconfined 'U' Tests--The deviator stress at failure for 'Q' tests has the same value for all lateral pressures for a given moisture content. Since no moisture is drained the strength corresponds to the value obtained on curve 'B'. 'U' tests are a special case with 'O' lateral pressure.

(1) Harvard University, Massachusetts Institute of Technology, Waterways Experiment Station, Vicksburg, Mississippi.

B. For Quick Consolidated 'Q_c' Tests--The water content, 'W_c', at end of preliminary consolidation under any selected lateral pressure may be obtained from curve A. Since the 'Q_c' test assumes that no change in water content occurs during the triaxial compression, the compressive strength $(\sigma_1 - \sigma_3)$ is taken from curve B at the same water content, 'W_c'.

C. For Slow Test (S)--The consolidated water content, 'W_c', corresponding to any selected stress $\sigma_3(=\sigma_2 = \sigma_1$ during preliminary consolidation) is determined from curve A. From this water content and stress, σ_3, curves A and B are followed in the direction of decreasing water content until a water content, 'W_f', at the maximum stress is found such that the value of σ_1 on curve A is equal to the initially assumed lateral pressure σ_3 plus the compressive strength $(\sigma_1 - \sigma_3)$ on curve B. For each assumed lateral pressure only one water content will give values on the two curves that satisfy this condition.

The stress circles and the corresponding rupture envelopes obtained from the above procedures have been found to be almost identical to those obtained from the best average test envelopes. Some deviation has been observed at the lower stress range for 'Q' tests on remolded compacted materials, where the rupture envelope was found to be parabolic. Remolded clays contain air in the voids and this gives the difference in the slope of the curves. It can also be shown that if curves A and B are parallel it follows that the Rupture envelope is a straight line passing through the origin.

7. ADVANTAGES OF THE WORKING HYPOTHESIS

The working hypothesis has four important advantages over previous methods of presenting triaxial data.

(a) With additional confirmation the above method can be used to eliminate the time consuming 'S' triaxial test, the drained strength being obtained from curves A and B.

(b) The presentation of strength results with the two curves does not involve the angle of internal friction nor the cohesion value. There is some doubt that the angle of internal friction has any significance for a cohesive soil since the values obtained from the Mohr's Diagram involve failure stresses obtained from materials not in the same condition. The moisture contents, densities and void ratios are not the same

with the different lateral pressures used in the tests. There is therefore small justification in assuming these conditions as the same and drawing a Mohr Rupture line tangent to the stress circles obtained in this manner.

The two-curve method does not involve the doubtful quantities, 'C' and '$\emptyset$', and is therefore much superior in this respect.

(c) Curves A and B are plotted directly from test data and show immediately the effects of initial differences in water content and difference of material.

(d) Curve A has been shown to be independent of the neutral stresses (pore pressures) and can be used with curve B to determine the effects of limiting pore pressures on the strength of the soil.

8. VARIABLES AFFECTING STRENGTH

Rutledge lists the important variables which affect the compressive and shearing strength of a soil. They are as follows:

A. Soil type, character and plasticity
B. Condition of soil--e.g. undisturbed, remolded, compacted, etc.
C. Stress history of the soil
D. Type of test -- Q, U, Q_c, or S
E. Minor principal stress during consolidation and during test
F. Initial water content
G. Initial void ratio or dry density
H. Initial degree of saturation
I. Water content at end of test at maximum stress
J. Void ratio or dry density at end of test at maximum stress
K. Degree of saturation at end of test

These variables do not influence all soils alike and a classification of cohesive saturated soils based on test results may be made.

Group A

Compressive strength varies as a direct function of water content at the end of tests and is not significantly affected by other variables except initial water content and plasticity of soil.

Group B

Compressive strength varies with water content at end of test and with minor principal stress.

Group C

Compressive strength varies with final water content, with minor principal stress and with density, plasticity and original character of specimens.

9. PARTIALLY SATURATED COHESIVE SOILS

At the time of the Rutledge report very little data was available on partially saturated cohesive soils. The following information is however given:

(a) Factors other than the water content at the end of test influence compressive strengths.

(b) The test lateral pressure appears to have almost an equal effect on the compressive strength as does the moisture content at the end of test.

(c) The type of test, (Q, Q_c, or S) has no consistent order or effect on the compressive strengths. 'Q' and 'Q_c' tests give average results which are about the same. However, slow tests were found to give sometimes greater and other times smaller values of compressive strength than the 'Q' and 'Q_c' tests. Soil type appears to govern such cases.

(d) A general tendancy has been noted for greater initial dry unit weights to give greater strengths under any given lateral pressure.

(e) Effects of compaction are indefinite.

(f) Final densities and degrees of saturation have marked influences on test strengths.

(g) The variables listed in paragraph 8 all have some influence on the compressive strength.

10. RECOMMENDATIONS FOR FUTURE TESTS

Partially saturated cohesive soil should be given top priority in future investigations.

PART II

INVESTIGATION OF CERTAIN STRENGTH-MOISTURE RELATIONSHIPS OF PARTIALLY SATURATED COHESIVE SOILS

1. INTRODUCTION

The Department of Transport of Canada, during 1945 and 1946, conducted an extensive soil survey on a bearing load evaluation program of certain airports. Methods of design used by most of the United States authorities were considered too conservative and, to justify the lighter construction of the Canadian airports, various test were made. These included all the usual soils classification tests as well as the California Bearing Ratio, the Housel Penetration, Cone Bearing, Bearing Plate and the Quick Triaxial and Consolidation tests.

The latter two tests were run on a great variety of cohesive soils most of which were partially saturated. Although the information was primarily for airport evaluation, the wide range and large number of tests performed made it possible to use the data for a study of the type recommended by Rutledge for partially saturated cohesive soils. (See S10, PI). The object of Part II is to determine whether a relationship similar to the two-curve method outlined by Casagrande for cohesive, saturated soils exists for the partially saturated soils examined under the D.O.T. test program.

2. GENERAL DESCRIPTION OF SOILS TESTED

Classification, consolidation and triaxial 'Q' tests were performed by the University of Alberta and the information was made available for this thesis. Undisturbed samples approximately 6 inches in diameter and 6 inches long were obtained in the field and shipped to the soils testing laboratory of the University. Soils were thus obtained from the airports shown in Table I:

Airport	U.S. Public Road Administration Classification	Average Liquid Limit	Average Plastic Index
Lethbridge	A-7, A-6[1]	39.5	20
Fort St. John	A-7	49.2	27
Grande Prairie	A-7	63.9	38.5
Saskatoon	A-7, A-6	46.5	23.7

Table I[2]

Soils from Ft. Nelson, Regina, Winnipeg, Calgary and Moncton airports were also tested at the University. The Ft. Nelson data did not include any triaxial tests and very few tests were performed on the Regina, Winnipeg and Moncton airports. The Calgary soils were non cohesive and do not belong in a discussion of cohesive soils. For these reasons only the soils listed in Table I have been included.

Dr. McLeod[3] in giving a general description of the soils states that the soils were well above the ground water table and only 21.7%[4] were found to be saturated. Little if any compaction was used during construction. Table I has been reproduced from his report.

3. LIMITS ON THE USE OF THE D.O.T. DATA

The test information was primarily obtained for load evaluation of airports and not as part of a triaxial test research program. Definite limits are thus set on the use of the data by the very nature of the soils tested and the tests performed.

a. The samples were obtained from under airports already built. They may have been subjected to remoulding, compaction, etc. Layers

(1) The Public Roads Administration describes these soils as follows:
A-6-- Fine grain soils, highly colloidal types of inorganic clays, medium to high plasticity.
A-7-- The great majority of inorganic clays, (e.g. glacial clays), and some types of organic clays, medium to high plasticity.

(2) Data from Dr. McLeod's Report, Airport Evaluation in Canada published by the U.S. Highways Research Board, Bulletin No. 4B, 1947.

(3) Dr. N.W. McLeod, Engineering Consultant, Department of Transport, Ottawa, Canada.

(4) Includes all airports except Calgary.

of overburden may have been removed or a section may have been completely filled material. It is impossible to estimate the stress history of the soil. If available this information could be used in checking the virgin compression curve of the consolidation test. Differences in consolidation curves may be due to different degrees of preconsolidation rather than to different soil types and a check would be valuable. The samples are thus undisturbed only when considered in respect to their position beneath an existing airport and not to the state they may have existed originally.

b. Airport loadings are mostly of a very rapid nature, for example, the live load and impact of an aircraft on landing or taxiing. A strength test for an airport evaluation program would have to duplicate field conditions with no time allowed for consolidation. Therefore, all the triaxial tests that were performed were of the 'Q' type, and no information was available to verify the 'Q' and 'S' test relationship to the consolidation curve. It may be pointed out that in this respect, the Casagrande two-curve hypothesis applies to all three types of test, 'Q', 'Q_c', and 'S'.

c. A considerable lapse of time occurred between obtaining the samples and the testing during which some drying of the samples was unavoidable. This drying may be quite high and combined with the fact the samples were not saturated in the field conditions, a comparison of a strength moisture curve to a consolidation curve in terms of percent moisture may not be possible. The consolidation samples are very nearly completely saturated and the two curves may not overlap sufficiently to establish any relationship. Table II gives the range of moisture contents obtained for the four airports in both the triaxial and consolidation tests.

Triaxial and Consolidation Test Moisture Contents

Location of Samples	Triaxial Test W%	Consolidation W%
Grande Prairie	15-32	20-58
Lethbridge	6-20	18-43
Fort St. John	14-28	18-36
Saskatoon	18-38	20-40

Table II

Comparison is not entirely justified even when the moisture contents are the same in the two tests. With a partially saturated triaxial sample and a completely saturated consolidation sample at the same moisture content it follows that the triaxial sample has the greater void ratio. The degree of saturation and void ratio have been shown to affect the strength. For this reason it was decided to investigate the triaxial and consolidation test data, first separately, and then for interrelationship.

d. Samples for the airport evaluation program were necessarily from different locations and depths on the airport to enable a general evaluation of the load bearing capacity. From a uniformity standpoint such sampling is highly undesirable and considerable variation was to be expected. Before the data could be used for the investigation a complete classification was necessary. The plasticity chart offered the most convenient method for such classification.

e. The Casagrande two-curve working hypothesis for saturated cohesive soil is independent of the pore pressure. In a partially saturated soil, pore pressure is difficult to define. Both air and water may be considered carrying neutral stresses. Air is highly compressible and in a three-phase system a change in void ratio can take place without the loss of any water or air. It is possible for the effective (intergranular) stresses to be mobilized almost instantaneously on the application of a load increment. On a saturated sample a similar load increment may produce failure since the increase in total stress is taken by the water phase without an increase in shearing resistance. The D.O.T. testing did not include any pore pressure measurements and their effect cannot be ascertained.

4. TREATMENT OF DATA

a. Original Arrangement of Data--The test data were originally arranged for each airport according to the location and depth along base lines established on the airport. The information was also separated for base course, sub-base (Fort St. John only), and subgrade materials. A code system was established using the following notation:

QL, QU, XJ, XE--for Lethbridge, Grande Prairie, Fort St. John and Saskatoon, respectively.

1U, 2U, ------- 22U -- for position along the base lines.

SG, BC, SB ----- subgrade, base course and sub-base, respectively

0-6, 6-12, 12-18 ----- depth in inches of sample below surface of base course, subgrade or sub-base

A sample of subgrade material from Lethbridge would have have been labelled as follows:

QU - 15U SG - 6-12

b. Classification of Test Results--It was apparent that the arrangement described in 'a' was not a satisfactory means of classification since it did not take into account the soil type. The liquid limit and plastic index for each sample, plotted on the plasticity chart enable the grouping of soils with certain similar characteristics[1] and it was decided to use this method for the classification. It is not, however, a complete classification because stratification, fissures, root holes, compaction, remoulding, etc., affect the strength and are not indicated on the plasticity chart. This information was not available and the plasticity chart had to be relied on solely.

Liquid limit and plasticity index values can be influenced by the preparation of the samples for the tests[2]. The test procedures were standardized and it is believed that the values of the liquid and plastic limits were obtained with a minimum of variation.

Using all the available liquid limit and plastic index values plasticity charts were prepared for each airport and a best 'fit line' drawn. Samples for which triaxial test results were available were given a number. The number '1' was given to the sample with the highest value on the best 'fit line'. Points that were on the same line at right angles to the best 'fit line' were given identical numbers. This number was called the 'plasticity number'. Increasing plasticity numbers were given to the samples with lower plasticity values. The plasticity number has been shown on all the strength-moisture plots. Should any relationship have existed between strength and plasticity it would have resulted in some logical order or arrangement of the plasticity numbers but this was not the case. It was only the extremely high and low plasticities

(1) Classification and Identification of Soils by Arthur Casagrande, Publication No. 432, 1947-48, Harvard Soil Mechanics Series.

(2) Control tests performed by the University of Alberta

that had an appreciable effect on the strength of the soils tested.

Fig. 2 shows the best 'fit line' of all the plasticity charts. Comparison with the standard 'A' line indicates that most of the soils fall above the 'A' line. Each airport gave a well defined best 'fit line' which indicates that the soils for each airport have the same geological origin(1). There is a difference between airports which was considered sufficient to prevent comparison of soils between different airports. The consolidation tests were also arranged in order of the position of the soils on the plasticity chart. In this case plasticity did not appear to make an appreciable difference and numbers similar to those used in the triaxial data were not used. The original computed consolidation data were in the form of void ratio and pressure in pounds per square inch. It was necessary to recompute all the values on a percent-moisture and pressure in kgm/cm^2-basis to enable comparison with the triaxial test data. (See Sample Calculations--S-9, P-II). Another difficulty arose in classifying both the triaxial and consolidation data according to the plasticity. Samples from depths 9-15" and 10-16" had no liquid and plastic limits. The values for 6-12" and 12-18" have been used depending on which appeared to be the more applicable but it is certain that a certain error could not be avoided. On the summary data sheet the figures in the brackets above the sample code number indicate from what depth liquid and plastic limit samples were obtained.

c. Graphs--The following graphs were drawn for each airport:

1. Plasticity Chart

2. Percent Moisture (arithmetic scale) versus Major Principal Stress (logarithmic scale) at failure. Separate plots were made for the lateral pressures of 0, 15 and 30 psi.

3. Percent Moisture (arithmetic scale) versus Major Principal Stress (logarithmic scale) at failure. The three lateral pressures have been shown on the same plot.

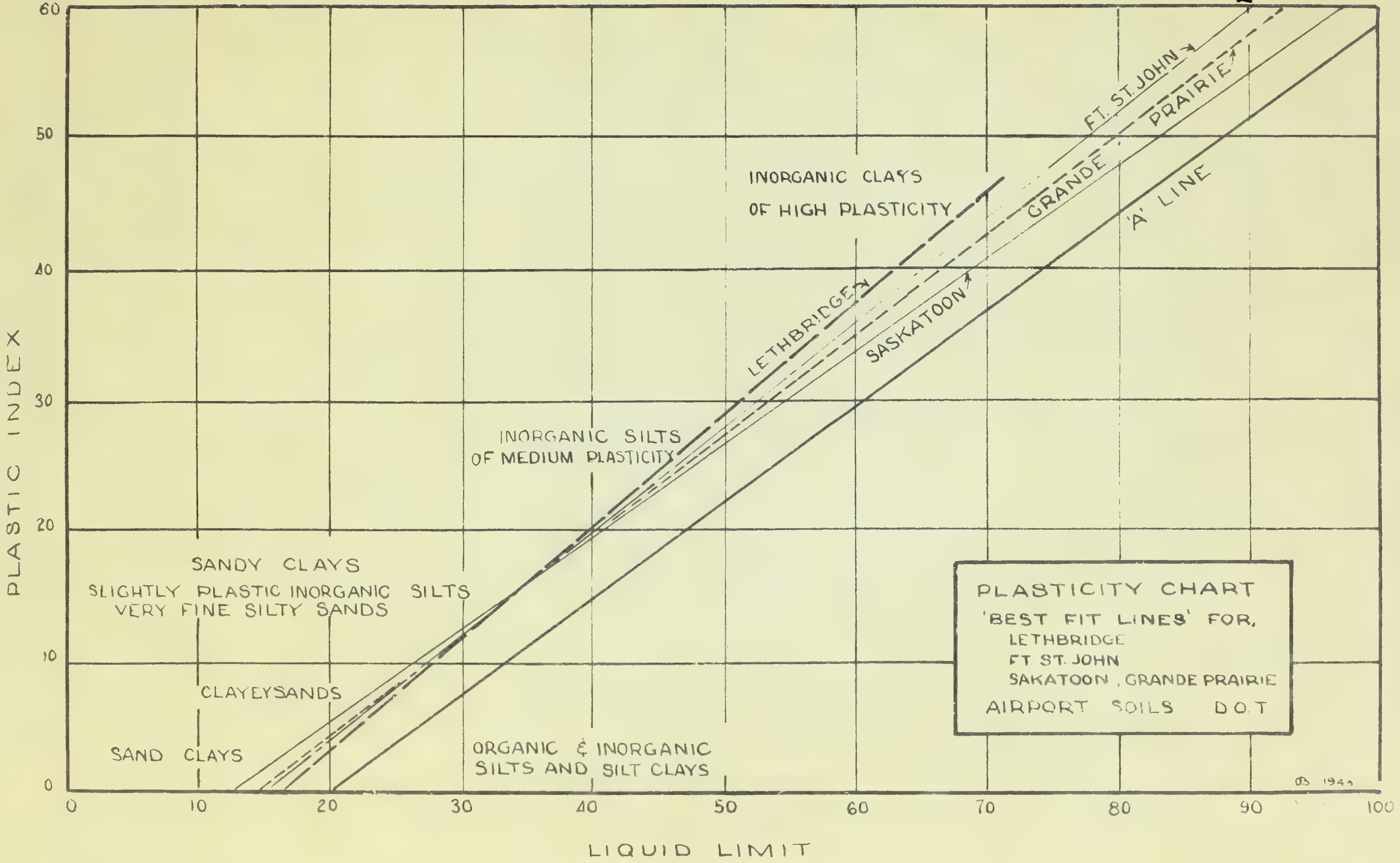
4. Percent Moisture (arithmetic scale) versus maximum Deviator Stress (logarithmic scale) at failure for 0, 15, and 30 psi lateral pressures combined.

5. Percent Moisture (arithmetic scale) versus Major Principal Stress (logarithmic scale) of the consolidation test. Superimposed on these graphs were the best 'fit curves' from the percent moisture versus maximum deviator stress at failure.

d. Selection of Graphs--In selecting the graphs to be plotted it was found that almost a straight line was obtained for the percent moisture content versus the major principal stress at failure on a semi logarithmic graph for the ranges of moisture content and soils investigated. The plot of percent moisture versus maximum deviator

(1) The theory has been advanced that soils from the same geological origin fall on a single line on the plasticity chart.

FIGURE 2


PLASTICITY CHART
'BEST FIT LINES' FOR,
LETHBRIDGE
FT. ST. JOHN
SAKATOON , GRANDE PRAIRIE
AIRPORT SOILS D.O.T
INORGANIC CLAYS
OF HIGH PLASTICITY
INORGANIC SILTS
OF MEDIUM PLASTICITY
SANDY CLAYS
SLIGHTLY PLASTIC INORGANIC SILTS
VERY FINE SILTY SANDS
CLAYEYSANDS
SAND CLAYS
ORGANIC & INORGANIC
SILTS AND SILT CLAYS
FT. ST. JOHN
GRANDE PRAIRIE
'A' LINE
LETHBRIDGE
SASKATOON
PLASTIC INDEX
LIQUID LIMIT
0
10
20
30
40
50
60
70
80
90
100

stress (semi logarithmic) tended to be more curved. To reduce the data to the simplest form, the relationship giving the straighter line was selected to enable easier evaluation of the effect of differences in initial void ratio, degree of saturation and plasticity. Using the major principal stress instead of the deviator stress made separate plots necessary for each lateral pressure. However, this was desirable since it eliminated crowding. For each point plotted, the other factors which were believed to influence strength as well as the code number were noted, for example, plasticity number, void ratio at start and the initial degree of saturation.

The graphs for 0, 15 and 30 psi lateral pressure were then combined to permit comparison. To reduce the information to a form comparable to the two-curve hypothesis the deviator stress at failure versus moisture content plots were made. Differences due to lateral pressures were noted and a best 'fit curve' obtained for each lateral pressure.

All the consolidation curves were then plotted and the best 'fit curves' of deviator stress at failure versus moisture content were superimposed.

e. Data-- The triaxial and consolidation test data have been condensed and presented in summary data sheets. Plastic limits, plasticity indices and specific gravities have also been included and the entire data arranged in order of position on the plasticity chart.

The compressive index, pre-consolidation pressure and swelling pressure from the consolidation test are not included. Consolidation test pressures were not considered sufficiently high to give accurate compressive indices and the other two quantities may be readily found from the data and graphs. They were not considered relevant to the discussion and thus were omitted.

5. DESCRIPTION OF TESTS

a. Specific Gravity--The method outlined in the University of Alberta Soils Testing Manual[1] was used.

(1) Testing Manual for Soils used in the undergraduate and graduate courses in Civil Engineering Soil Mechanics, 1948.

b. Plastic Limit--The A.S.T.M., (American Society for Testing Materials), method D424-39 was used.

c. Liquid Limit--The A.S.T.M. method D423-39 was used.

d. Triaxial Compression Tests--The procedure outlined for 'Q' tests in the University of Alberta Soils Testing Manual was used. Specimens were cut from larger undisturbed samples using a cutting tube. Briefly, the tests consisted of loading to failure, 1.4-inch diameter by 3 to 4-inch long samples at 0, 15 and 30 psi lateral pressures in a closed chamber having a transparent lucite wall. No allowance was made for drainage and the load increments were applied by a jack arrangement and transmitted by a loading arm, piston and tilting head onto the sample. The sample was seated on a pedestal which in turn rested on a scale platform enabling the measurement of the total load on the sample. The triaxial testing machine was thus of the reaction type. Air was used for the lateral pressure medium and the samples at 15 and 30 psi lateral pressures were enclosed in rubber sleeves. Moisture content readings, measurements, etc., and the rates of loading, load increments, etc., have been shown on the sample data sheet on Page .

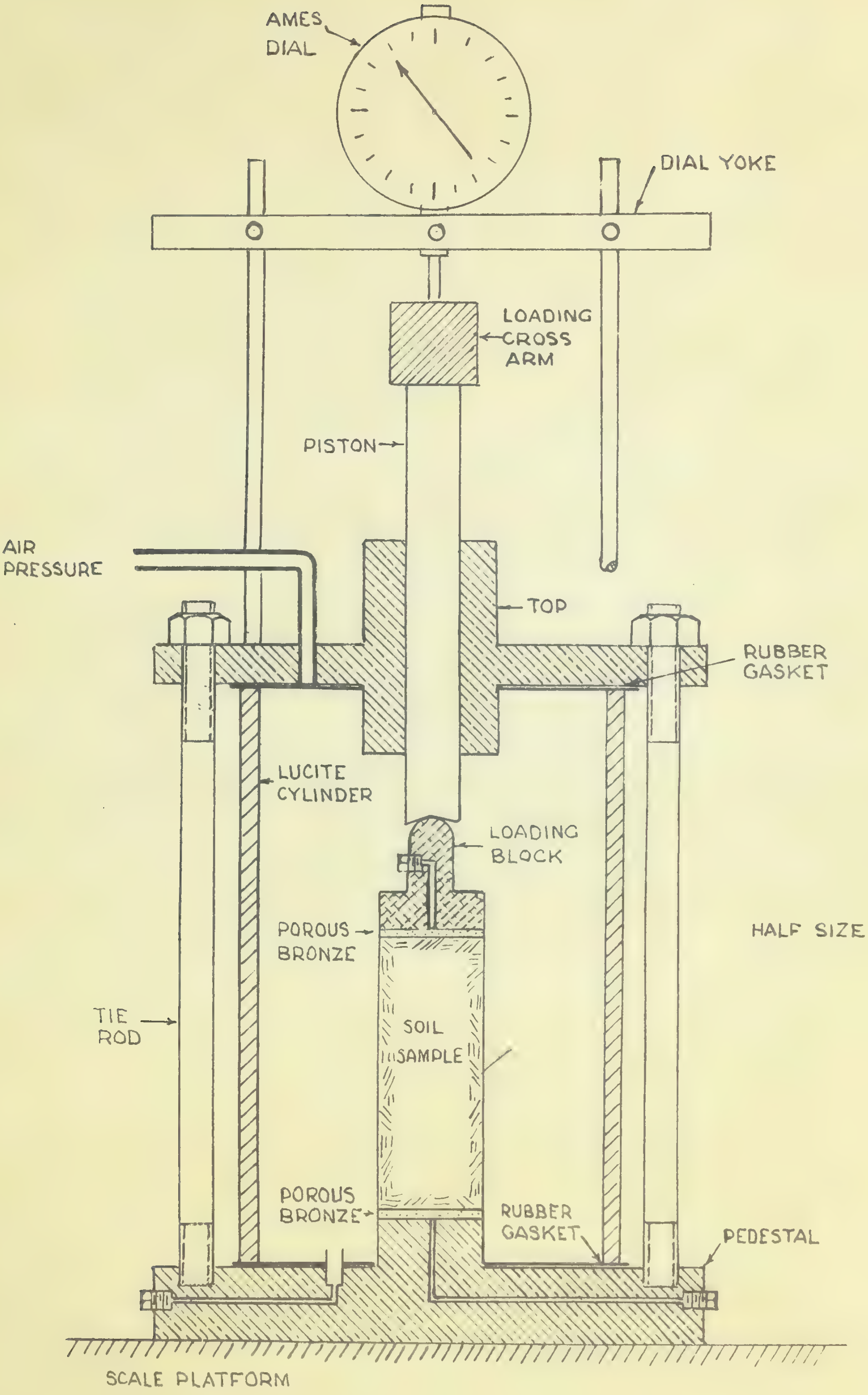
e. Consolidation Tests--The University of Alberta Soils Testing Manual procedure was used. Rates of loading, dimension of sample, etc., have been indicated on the sample data sheet, Page 30.

6. DIAGRAMS AND PHOTOGRAPHS OF EQUIPMENT

Diagrammatic representations of the equipment have been given on Page 17 and Page 18. Photograph 2 shows one of the testing machines used in the D.O.T. triaxial tests and photographs 1A and 1B show the consolidation equipment used.

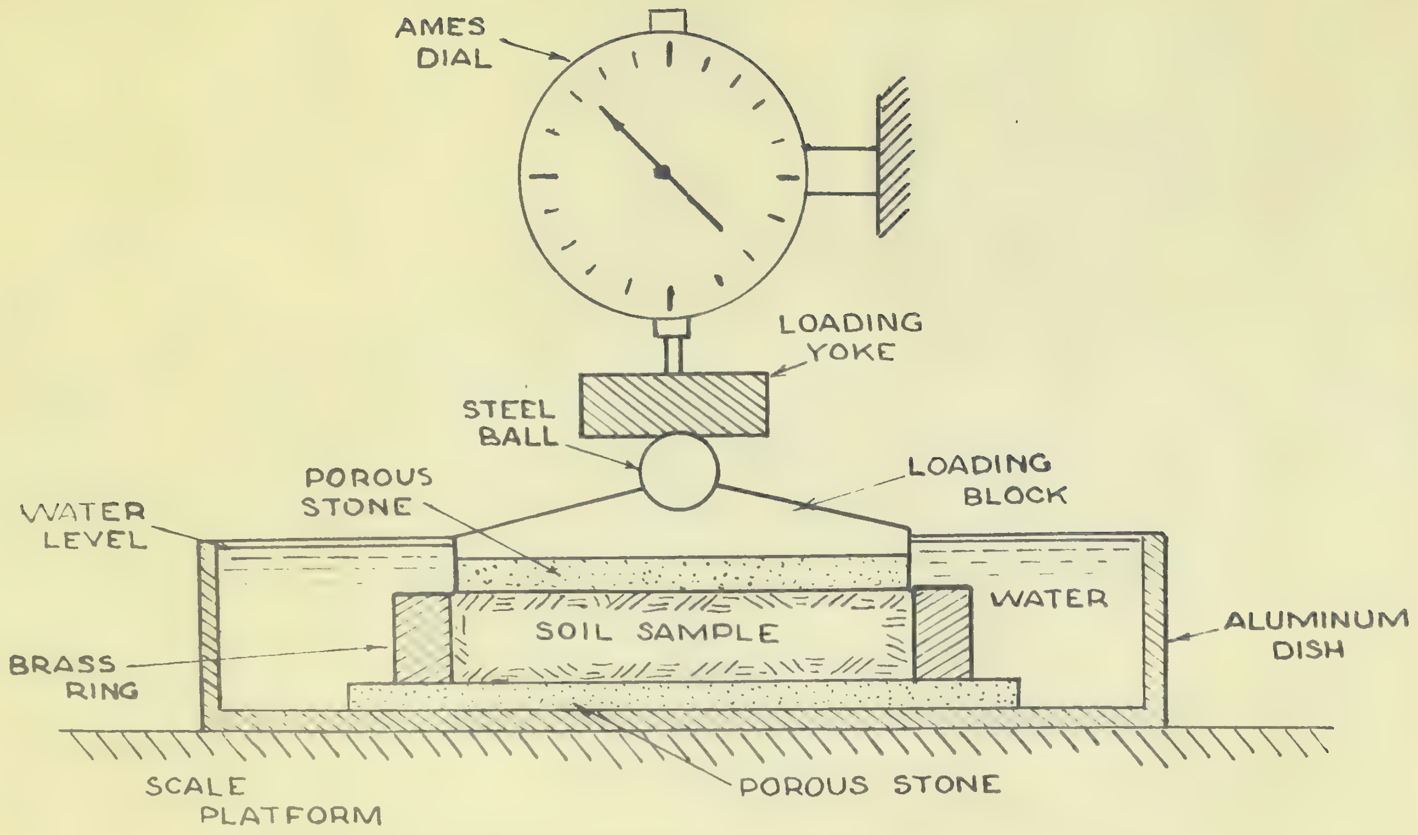
7. RELIABILITY OF DATA

At the time the tests were made, apparatus and techniques had not been developed to the extent mentioned in the Rutledge report. New laboratory personnel had to be trained and some error due to inexperience may have been introduced. For example, final moisture contents were not obtained for a number of triaxial and consolidation tests. It was impossible to use this information on the present basis of computation. Probable errors due to apparatus and technique may



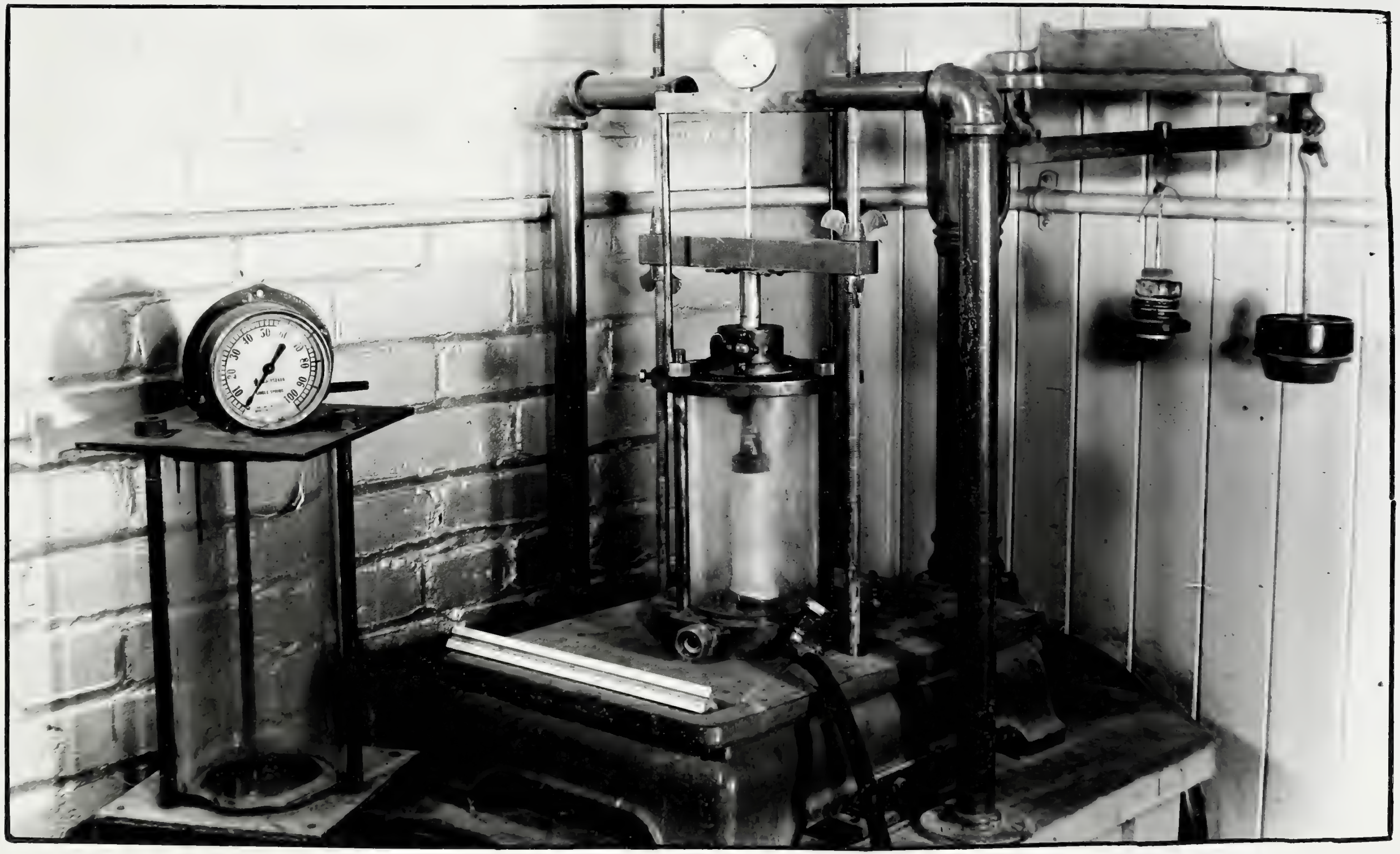
TRIAXIAL COMPRESSION TEST APPARATUS

Fig. 3



CONSOLIDATION TEST APPARATUS

FIG. 4



TRIAXIAL COMRESSION APRARATUS

PHOTOGRAPH IB

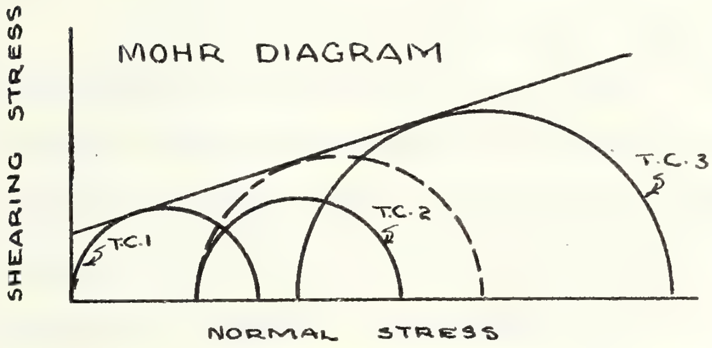
PHOTOGRAPH IA

CONSOLIDATION TEST APPARATUS

be computed and were taken into account in selecting suitable scales for the graphs.

Non-uniformity of samples presents an unascertainable source of errors. In determining 'C' and 'Ø' it was pointed out in Part I that even the test procedure caused non-uniformities which made the value of these quantities doubtful. Stratification and fissures may cause premature failure of triaxial samples. Rough handling in transit, and for some samples the ordinary preparation for the tests, damaged samples and gave lower compressive strengths. An occasional compressive strength appears high and may be due to over lubrication or dirt on the piston.

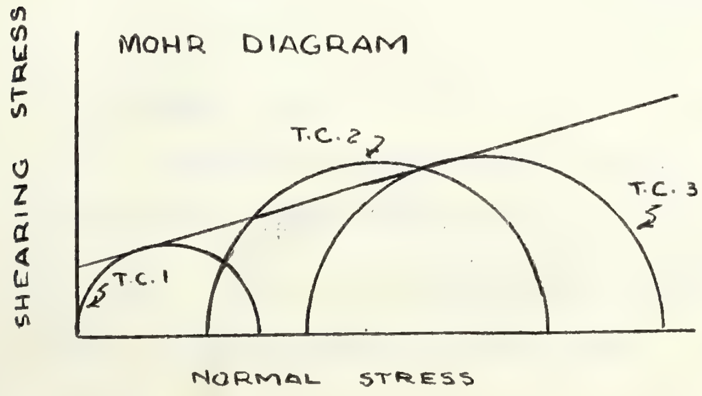
Where the source of error could definitely be determined a correction was made, for example, in the following triaxial data, Fig. 5 .



Test No.	Moisture Content at end %
TC_1	12.6
TC_2	12.6
TC_3	12.6

Fig. 5

The compressive strength $\sigma_1 - \sigma_3$ for test TC_2 appears low. The quantities which affect strength were found to be similar for TC_1, TC_2, and TC_3. A correction is justified and is indicated by the broken circle. Such corrections were found to plot much better on the strength moisture plots. In Fig. 6 no correction should be made.



Test No.	Moisture Content at end %
TC_1	13.5
TC_2	13.0
TC_3	13.5

Fig. 6

Sample TC_2 differs in moisture content and does not belong in the same group as TC_1 and TC_3. At least two circles must have had the same moisture contents, void ratios, etc., before 'C' and 'Ø' could be found.

Increased moisture contents during the triaxial test were noted for some triaxial tests. An error in weighing is the only possible explanation since no moisture was added. Sample 13 USG 9-15, Lethbridge for which a sample data sheet is given on Page 30 , is an example. Several other triaxial tests for the Lethbridge soils show similar errors. Sample 13 USG 9-15, Lethbridge also shows the effect of improper seating of the loading block, piston, loading cross beam or Ames dial. Until proper contact was obtained between these parts, initial dial reading increments were too large. See Fig. 3 . Percent strains in such cases were computed from the dial readings after good contact had been obtained.

The consolidation tests were computed on the basis of 100% saturation at the end of the test. This assumption may not be entirely correct regardless of the length of time the sample may have been allowed to soak. For the lower pressure range, saturation probably did not occur for most samples but the curves are not of particular interest in this region.

A few cases were noted where the loads were changed too rapidly on the consolidation test and accounted for the irregularities on the W% versus pressure curves.

8. MEMBRANE EFFECT

Rutledge points out that the membrane effect on the triaxial test strengths depends on the type of soil as well as the type of membrane used.

Example Soil	Increase in Compressive Strength
Massena Clay	0.18 kg/cm^2
Chicago Clay	0.08 kg/cm^2

The above membranes were 0.04 cm. thick. Membranes 0.006 cm. thick were also used and found to give negligible differences. The D.O.T. data were plotted without any correction for membranes but their effect has not been neglected. In the comparison of

strength-moisture curves a correction was established for the best 'fit lines'. Since no membrane calibrations were made, an arbitrary value of 0.10 kgm/cm^2 was subtracted from the compressive strengths obtained with the 15 and 30 psi lateral pressures.

9. SAMPLE COMPUTATIONS

A. Specific Gravity, Liquid Limit, Plastic Limit--The specific gravity, liquid and plastic limit computations are shown on the sample data sheet, (Pages 26 to 29) and require no further explanation. Probable errors were computed for similar tests performed by students in the university soils testing laboratory. The following values may be taken as representative of careful work:

Quantity	Probable Error %
Specific Gravity	$\mp$ 0.5%
Liquid Limit	$\mp$ 1.0%
Plastic Limit	$\mp$ 1.0%
Plastic Index	$\mp$ 2.5%

B. Triaxial Compression

i. Moisture Contents--Computations for the moisture contents are shown on the sample data sheet Page 30 .

ii. Vertical Axial Strain and Stress--The computations for vertical axial strain and stress are as follows:

e.g., example TC_2 QL 13 USG 9-15 at failure

Load on pan	=	400 gm.
Load on pan x scale factor $\frac{400x50}{1000}$	=	20.0 kgm.
Weight of block and piston	=	0.4 kgm.
Total = 20.0 + 0.4	=	20.4 kgm.
Area of sample	=	9.77 cm^2
Dial increment(1) = 0.4930 - 0.4030	=	0.0900 in.
Length of sample	=	4.24 in.
Percent strain = $\frac{0.0900x100}{4.24}$	=	2.12%
100 - % strain	=	97.9

(1) See S-7, P-II

Corrected area $= \dfrac{9.77}{0.979} = 9.99\ \text{cm}^2$(1)

Deviator stress $= \dfrac{20.4}{9.99} = 2.03\ \text{kgm/cm}^2$

Lateral pressure $=$ 15 psi $= 1.05\ \text{kgm/cm}^2$

Total vertical axial stress at failure

$= 1.03 + 2.03 = 3.08\ \text{kgm/cm}^2$

It may be shown that this value may be as much as 10% too low. The last load before failure has been used to compute the compressive strength. With a final load increment of slightly less than the 50 gms. that were used, the sample might have withstood the increased vertical stress. The pan load may thus be approximately $\frac{50}{400} \times 100\%$ or 12.3% lower than the actual failure condition and consequently the deviator stress 20 x .123, or 0.25 kgm/cm^2 too low. If the air pressure of 15 psi was within $\mp$ 0.5 psi or the equivalent 1.05 kgm/cm^2 $\mp$ 0.04 kgm/cm^2, the vertical axial stress at failure was $\frac{0.04 + 0.25}{3.08} \times 100\%$ or very nearly 10% too low.

The percent moisture at the end of the triaxial test can be determined much more accurately to within $\mp$ 1%. Very weak soils may have compressive strengths indicated that are 50% or more, less than the actual values. The scales for the strength-moisture relationships graphs were thus selected accordingly. The strength scales were more compressed to take into account the large probable error and best 'fit lines' were drawn to correspond to greater strengths where any doubt arose.

iii. Void Ratio, Degree of Saturation and Unit Weight at Start--

Volume of sample $= 9.77 \times 4.24 \times 2.54 = 105$ cc

Volume of solids $= \dfrac{\text{wt. of soil}}{\text{Sp. Gravity}} = \dfrac{150.5}{2.69} = 56$ cc

Volume of voids $= 105 - 56 = 49$ cc

Initial Void Ratio $= \dfrac{49}{56} = 0.875$

Volume of water at start $=$ 22.0 cc

Degree of saturation at start $= \dfrac{22.0}{49.0} \times 100\% = 44.9\%$

(1) This method of correcting the area is empirical. It has been used as standard practice at the University of Alberta and by many authorities.

$$\text{Corrected area} = \frac{\text{Original Area}}{1 - \text{strain}}$$

Unit weight at start

$$\text{dry basis} = \frac{150.5}{105} \times 62.4 = 89.5\,\#/ft^3$$

$$\text{wet basis} = \frac{172.5}{105} \times 62.4 = 102.5\,\#/ft^3$$

C. Consolidation Tests

The computations have been shown in tabular form on the sample data sheet Page 32. They differ from the original calculations made for the D.O.T. in that % moistures are determined instead of void ratios and also the pressures are in kgm/cm^2 instead of #/sq.in. Complete saturation at the end of the test and the fact that dial readings were proportional to the change in moisture content were assumed. Although the swelling pressures were not used, the method of obtaining them has been indicated on graph 3 .

10. SAMPLES OF ORIGINAL DATA SHEETS AND GRAPHS

Sample data sheets used in the D.O.T. program are shown on the following pages. They include:

1. Specific Gravity Determination
2. Liquid Limit and Flow Chart
3. Plastic Limit
4. Triaxial Compression 'Q' Test
5. Consolidation Test and W% versus Pressure graph.

DATA SHEET

Specific Gravity Determination (A)

Sample . Q L 11 USG 0-6 . . . . Date . 31/12/45 . . . .

. . . SUBGRADE 0-6 . . . . Technician . P.F.D. . . . .

Description of Sample . . . .

. . . .

. . . .

Sample No.		1
Flask No.		A
Wt. Sample Wet + Tare	-gms.	
Wt. Tare	-gms.	
Wt. Sample Wet	-gms.	
Wt. Sample Dry + Tare	-gms.	
Wt. Sample Dry (W_s)	-gms.	80.00
Tare No.		
W_{bws}	-gms.	711.70
Temperature	T°C	22.6
W_{bw}		661.67
S_s		2.67

$$S_s = \frac{W_s}{W_s + W_{bw} - W_{bws}} = \frac{80}{29.97}$$

Remarks . . . .

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LIQUID LIMIT

SAMPLE . QL - 11 USG 0-6 . . . . DATE . 3/2/46 . . . .

. . . . LETHBRIDGE N.E. - S.W. 22 + 05 ℄ . . . . TECHNICIAN . . . A.M. . . . .

DESCRIPTION OF SAMPLE . . . .

. . . .

. . . .

Trial No.	1	2	3	4		
No. of blows	23	22	3	3		
Container No.	54 A	79	73	84		
Wt. Container + Wet Sample - gms.	87.92	103.25	83.87	89.17		
Wt. Container + Dry Sample - gms.	86.23	101.77	81.62	87.53		
Wt. Container - gms.	81.25	97.45	76.51	83.85		
Wt. of water - gms.	1.69	1.48	2.25	1.64		
Wt. of Dry Soil - gms.	4.98	4.32	5.11	3.68		
Moisture Content %	33.9	34.3	44.0	44.6		

REMARKS . . . .

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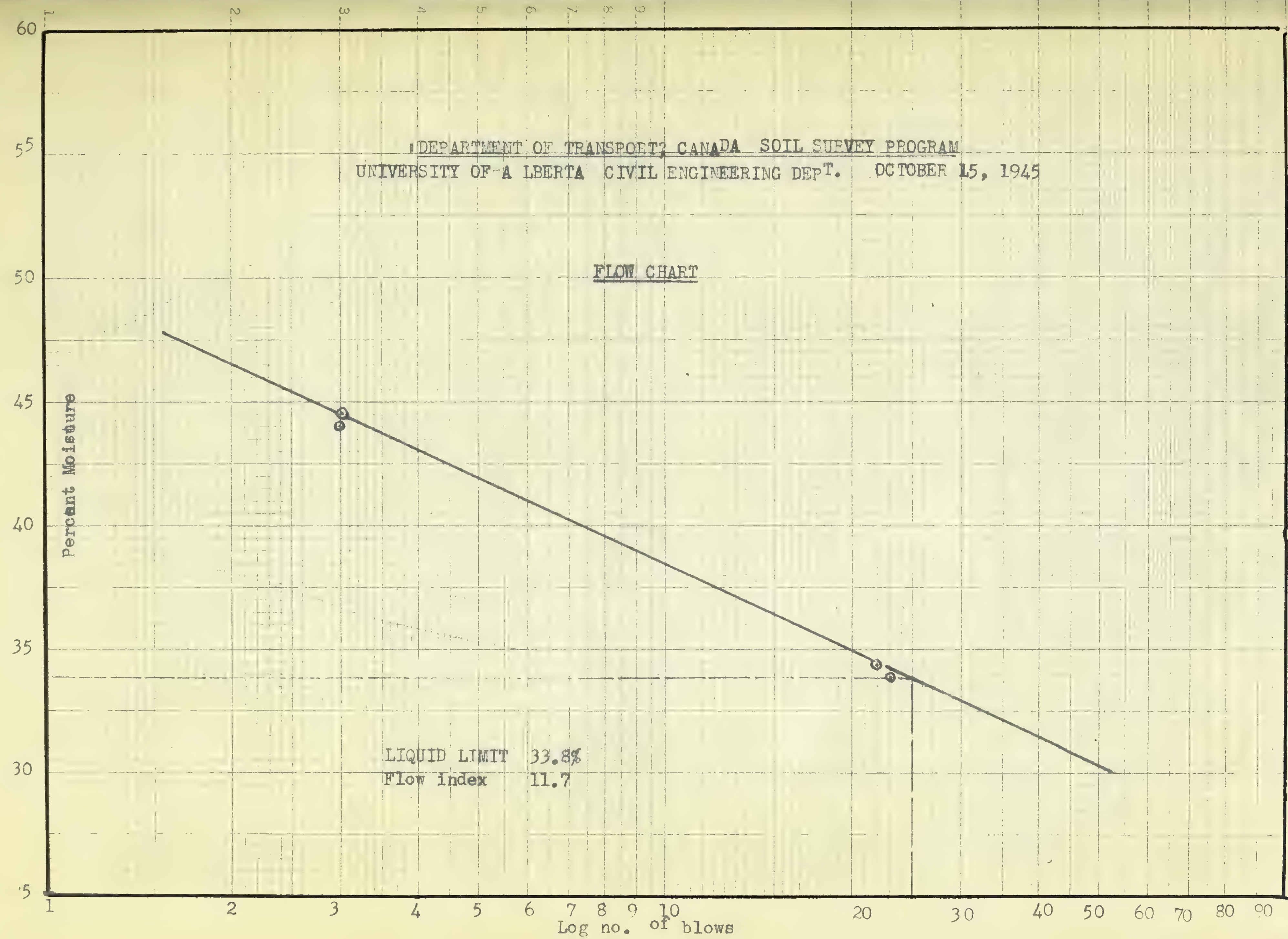
DEPARTMENT OF TRANSPORT, CANADA SOIL SURVEY PROGRAM
UNIVERSITY OF ALBERTA CIVIL ENGINEERING DEPT. OCTOBER 15, 1945
FLOW CHART
LIQUID LIMIT 33.8%
Flow Index 11.7
Percent Moisture
Log no. of blows
60
55
50
45
40
35
30
25
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90

DATA SHEET

PLASTIC LIMIT

SAMPLE . . Q L 11 USG 0-6 . . . . DATE . 3/2/46 . . . .

. . . . TECHNICIAN . A.M. . . . .

DESCRIPTION OF SAMPLE . . LETHBRIDGE . . NE - SW 22 + 05 ℄ . . . .

. . . . SUBGRADE . . . .

. . . .

Sample No.	1	2		
Container No.	4	24		
Wt. Container + Wet Sample - gms.	22.155	21.140		
Wt. Container + Dry Sample - gms.	21.958	20.856		
Wt. Container - gms.	21.007	19.463		
Wt. of Water - gms.	0.197	0.284		
Wt. of Dry Sample - gms.	0.951	1.393		
Moisture Content %	20.7	20.4		

AVERAGE PLASTIC LIMIT = 20.6 %

PLASTICITY INDEX = $W_L - W_P$ = 33.8 - 20.6 = 13.2

TOUGHNESS INDEX =

REMARKS . . . .

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TRIAXIAL COMPRESSION TEST

Date Oct 14/45

Sample 15USG 9-15 QL LETHBRIDGE

Technician AJ Test for D.O.T.

Machine no. 3 Load Factor 50 Wt. Piston & Block ____ gm.

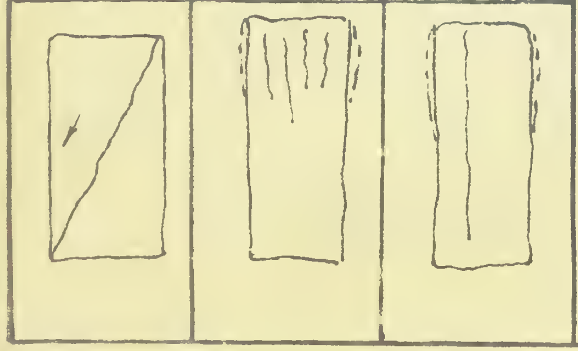
Specific Gravity 2.69

	T.C. 1	T.C. 2	T.C. 3
Lateral Pressure p.s.i.	0	15	30
Dia. Sampling Tube in.	1.39	1.39	1.39
Area Sampling Tube cm.	9.77	9.77	9.77
Length of Sample in.	2.91	4.24	3.86
Wt. Tube + S + W at start gm.	269.1	348.9	333.4
Wt. Tube gm.	145.03	176.40	175.90
Wt. S + W at start gm.	124.1	172.5	157.5
Wt. S + W + Dish at end gm.	196.1	261.9	201.6
Wt. S + Dish gm.	180.6	238.8	179.2
Dish no.	A	B	C
Wt. Dish gm.	72.78	83.34	40.74
Wt. Soil gm	107.8	150.5	138.5
Wt. W at start gm.	15.3	22.0	18.0
Wt. W at end gm.	15.5	15.3	16.1
% Moisture start	14.2	14.6	13.7
% Moisture end	14.4	15.3	16.1
Degree Saturation start %	41	45	43
Unit weight lb./ft.			

Load increment applied on the min. Rdg. taken ½ min. later

SKETCH AT FAILURE

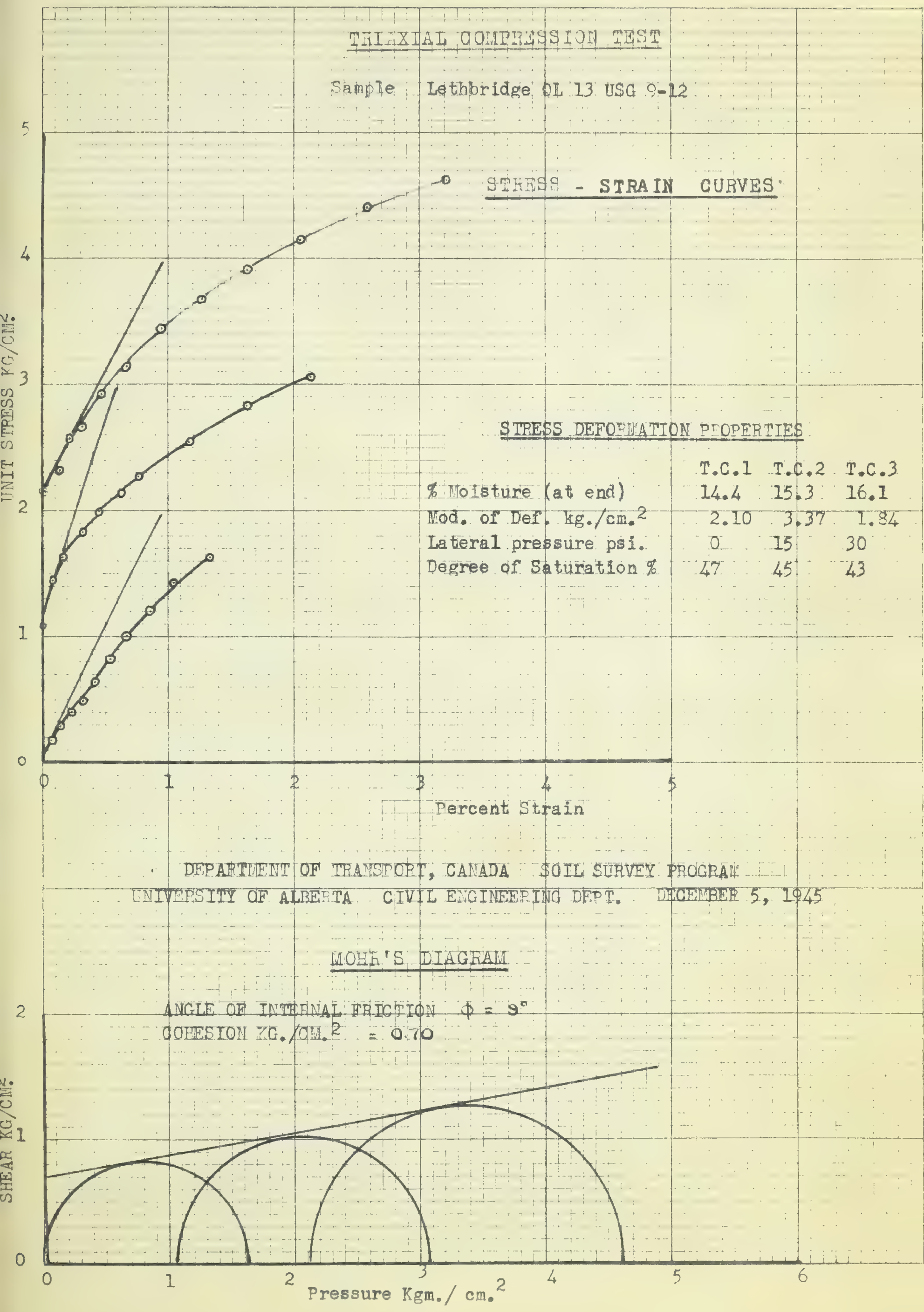
T.C. 1 T.C. 2 T.C. 3



REMARKS

T.C. 1 Load on Pan gm.	T.C. 1 Dial Rdg. in.	T.C. 1 Strain %	T.C. 1 Area cm.^{2}	T.C. 1 Axial Stress Vert. kg/cm^2	T.C. 2 Load on Pan gm.	T.C. 2 Dial Rdg. in.	T.C. 2 Strain %	T.C. 2 Area cm.^{2}	T.C. 2 Vert Axial Stress kg/cm^2	T.C. 3 Load on Pan gm.	T.C. 3 Dial Rdg. in.	T.C. 3 Strain %	T.C. 3 Area cm.^{2}	T.C. 3 Vert. Axial Stress kg/cm^2
0	.5270	0	9.77	0	0	.5160				0	.7180			
20	.5260	0.03	9.78	0.05	20	.5137				20	.7044	0	9.77	2.16
40	.5245	0.09	9.78	0.20	50	.4930	0	9.77	1.08	50	.7000	0.11	9.78	2.31
60	.5226	0.15	9.79	0.31	80	.4903	0.06	9.77	1.49	80	.6959	0.22	9.79	2.5[illegible]
80	.5201	0.24	9.81	0.41	110	.4869	0.14	9.78	1.64	100	.6920	0.32	9.80	2.6[illegible]
100	.5170	0.34	9.81	0.51	150	.4798	0.31	9.80	1.84	150	.6855	0.49	9.82	2.91
130	.5145	0.43	9.81	0.65	180	.4737	0.45	9.81	2.00	200	.6781	0.68	9.84	3.16
160	.5117	0.56	9.82	0.82	210	.4673	0.63	9.82	2.15	250	.6685	0.93	9.86	3.42
200	.5071	0.68	9.83	1.02	240	.4595	0.79	9.83	2.30	300	.6567	1.24	9.90	[illegible].63
240	.5016	0.87	9.85	1.22	290	.4442	1.17	9.88	2.56	350	.6415	1.63	9.93	3.91
280	.4960	1.07	9.87	1.43	350	.4245	1.61	9.93	2.86	400	.6245	2.07	9.97	4.16
320	.4873	1.36	9.90	1.62	400	.4030	2.12	9.98	3.08	450	.6040	2.60	10.03	4.41
380										500	.5805	3.21	10.09	4.64

GRAPH 2


TRIAXIAL COMPRESSION TEST
Sample Lethbridge OL 13 USG 9-12
STRESS - STRAIN CURVES
UNIT STRESS KG/CM.2
Percent Strain
STRESS DEFORMATION PROPERTIES
T.C.1 T.C.2 T.C.3
% Moisture (at end) 14.4 15.3 16.1
Mod. of Def. kg./cm.2 2.10 3.37 1.84
Lateral pressure psi. 0 15 30
Degree of Saturation % 47 45 43
DEPARTMENT OF TRANSPORT, CANADA SOIL SURVEY PROGRAM
UNIVERSITY OF ALBERTA CIVIL ENGINEERING DEPT. DECEMBER 5, 1945
MOHR'S DIAGRAM
ANGLE OF INTERNAL FRICTION φ = 9°
COHESION KG./CM.2 = 0.70
SHEAR KG/CM.2
Pressure Kgm./ cm.2

CONSOLIDATION TEST DATA AND COMPUTATION SHEET

Sample...Q.L. 11.USG. 0:6..
Description of sample......
.NE.-SW...22+05 ½.....
....Subgrade...0:6.......
Date......Jan. 1946......
Technician..................
Soil
S_s = ...2.68., H_s = 0.421 ins.
e (End) = ...
e (Start = ...
e (Start, Dimensions) = 0.762

Ring Data
Ring No....4..........
Weight. 235.34...gms.
Thickness.0.74...ins.
Diameter..2.63...ins.
Area.......35.02.cms.²

Machine Data
Machine No.........3....
X Factor ....100..
Wt. Block + Stone
+ Ball = ...gms.

Cons. Sample Weights
T+R +S+W (End) =.......—..gms.
T+R+S (End) =.......—..gms.
T (R)o....... =.......—..gms.
R+S+W (End) =.356.58..gms.
R+S+W (St.) =.344.41...gms.
R+S =..335.68..gms.
S =..100.34...gms.
Water (End) =.8.73 gms..= 8.7 %
Water (St.) = 20.90 gms..= 20.8 %
Water (Start Saturated)..21.9...= %

Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks
27/1/46 15:50	30m ·9937	29/1/46 10:00	30/1/46 10:15	31/1/46 8:36	1/2/46 8:36	2/2/46 8:25
10 GM.	1h 936	123 GM	254 GM	512 GM	1030 GM	2067 GM
0s ·9728	3¾h 934	6s ·9878	6s ·9800	6s ·9681	10s ·9524	6 ·9220
6 728	≠	10 877	10 799	10 679	15 520	10 195
10 728	28/1/46 13:00	15 873	15 798	15 675	30 500	15 180
15 728	58 GM.	30 871	30 790	30 670	1m 486	30 149
30 728	6s ·9922	1 870	1m 787	1m 662	2 472	1m 119
1m 728	10 919	2 869	2 781	2 658	4 460	2 090
water to bottom	15 918	4 865	4 773	4 650	10 441	4 062
2 ·9750	30 918	8 862	11 768	8 644	16 430	8 042
4 793	1m 916	15 861	15 767	15 639	31 415	15 020
8 846	2 913	31 858	30 761	30 631	66 397	30 ·8999
15 868	4 911	1h 855	1h 759	66 624	3h 374	1h 982
33 882	8 910	2½ 850	4½ 748	2 620	7¼ 353	2¼h 963
82 897	15 908	4 847	10 742	4¾ 612	11 348	9 941
water to top	32 905	8 842	22 737	7 609	23½ 341	15 933
2h ·9907	65 902	24 839		14¾ 603	≠	27½ 924
6 923	2⅓h 901	≠		24 600		≠
17½ 939	5½ 900			≠		
≠	21 898					
28/1/46 9:40	≠					
26 GM.						
6s ·9938						
10 938						
15 938						
30 938						
1 938						
2 938						
4 937						
8 937						
15 937						

Computations

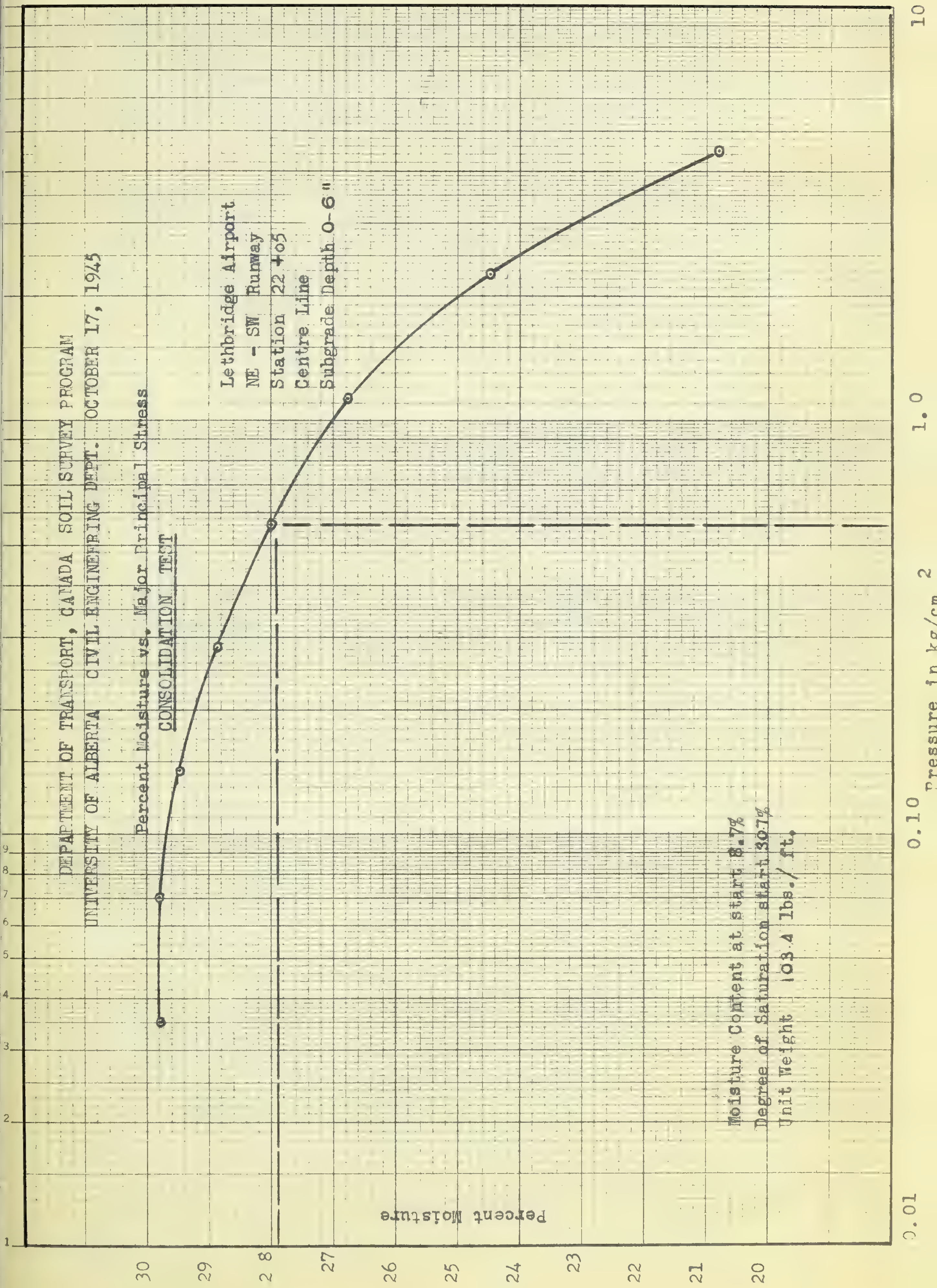
Computed by ....

Time Interval	Load on Pan (gms.)	Corr. Dial Rdg. (ins.)	Deflection (ins)	Defl.x100 / $S_s \times H_s$	W%	Pressure g/cm²
0	—	0.9728	0.0084	7.14	27.9	—
17½	10	.9939	.1015	9.02	29.8	0.035
3¼	26	.9934	.1010	8.96	29.8	0.070
21	58	.9898	.0974	8.65	29.5	0.141
24	123	.9839	.0915	8.12	28.9	0.282
22	254	.9737	.0813	7.22	28.0	0.563
24	512	.9600	.0676	6.00	26.8	1.128
23½	1030	.9341	.0417	3.70	24.5	2.25
7½	2067	.8924	.0000	0.00	20.8	4.50

$H_s = \left(\frac{S}{S_s \times \text{Area} \times 2.54}\right)$ ins.

$W = \text{previous } W \pm \frac{\text{Defl}}{H_s \times S_s}$

GRAPH 3


DEPARTMENT OF TRANSPORT, CANADA SOIL SURVEY PROGRAM
UNIVERSITY OF ALBERTA CIVIL ENGINEERING DEPT. OCTOBER 17, 1945
Percent Moisture vs. Major Principal Stress
CONSOLIDATION TEST
Lethbridge Airport
NE - SW Runway
Station 22 +05
Centre Line
Subgrade Depth 0-6"
Moisture Content at start 8.7%
Degree of Saturation start 30.7%
Unit Weight 103.4 lbs./ ft.
Percent Moisture
30
29
28
27
26
25
24
23
22
21
20
0.01
0.10
1.0
10
Pressure in kg/cm 2

11. <u>COMPUTED DATA SUMMARY AND GRAPHS</u>

The graphs selected in S-4C, P-II, are given in the order mentioned and for each airport. Consolidation test data includes sufficient information to compute void ratios if desired.

The code numbers used for the triaxial samples on the strength-moisture relationships need some explanation.

e.g. 20 - 0.850 - 82.0 - 5 USG - 0-6

Their significance is as follows:

Plasticity number--initial void ratio from measurement--degree of saturation % at start--sample No. in the order given.

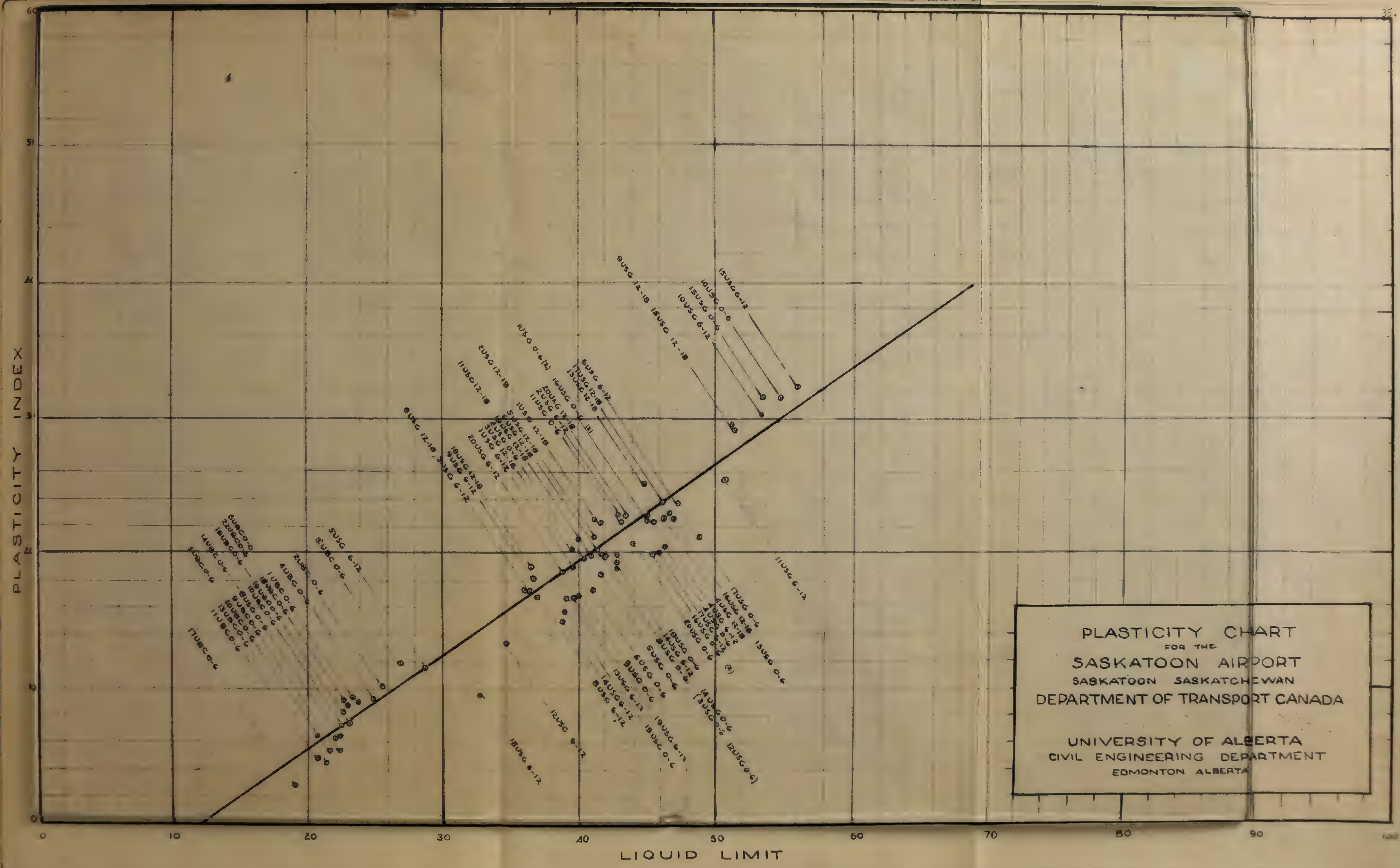
In the consolidation test data summary a discrepancy will be usually noted if the saturated moisture content is computed from the initial degree of saturation and initial moisture content and compared to the column of computed % moisture at start. Two explanations are possible:

a. The initial degree of saturation is based on measured length and the diameter of the sample which has a large probable error due to the small size of the sample.

b. The assumption of 100% saturation at the end of the test is not entirely correct.

The values of cohesion and angle of internal friction have been included in Part II to be used in Part III.

SASKATOON


PLASTICITY CHART
FOR THE
SASKATOON AIRPORT
SASKATOON SASKATCHEWAN
DEPARTMENT OF TRANSPORT CANADA
UNIVERSITY OF ALBERTA
CIVIL ENGINEERING DEPARTMENT
EDMONTON ALBERTA
PLASTICITY INDEX
LIQUID LIMIT
0
10
20
30
40
50
60
70
80
90

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED // Saskatoon

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED: Degree Saturation %	UNCONFINED: % Moisture Start	UNCONFINED: % Moisture End	UNCONFINED: Void Ratio Start	UNCONFINED: Deviator Str. Kgm/cm^2	15 p.s.i. LATERAL PRESSURE: Degree Saturation %	15 p.s.i.: % Moisture Start	15 p.s.i.: % Moisture End	15 p.s.i.: Void Ratio Start	15 p.s.i.: Max. Major Principal Stress Kgm/cm^2	15 p.s.i.: Deviator Stress Kgm/cm^2	30 p.s.i. LATERAL PRESSURE: Degree Saturation %	30 p.s.i.: Percent Moisture Start	30 p.s.i.: Percent Moisture End	30 p.s.i.: Void Ratio Start	30 p.s.i.: Max. Major Principal Stress Kgm/cm^2	30 p.s.i.: Deviator Stress Kgm/cm^2	'C' Cohesion Kgm/cm^2	Angle of Internal Friction
1	15 USG 9-15	56.0	32.1	2.70	96.4	34.2	33.8	0.946	0.56	96.7	34.4	34.2	0.956	1.84	0.78	97.8	34.4	34.0	0.954	2.91	0.80	0.28	5
2	15 USG 0-6	53.7	31.6	2.72	91.0	27.8	27.2	0.877	1.21	103	28.0	27.8	0.788	2.96	1.90	101.0	27.6	27.4	0.790	4.05	2.94	0.53	8½
3	16 USG 9-15	46.6	23.0	2.73	67.6	30.2	30.0	0.928	0.77	70.5	29.1	28.9	0.956	1.98	0.92	70.7	29.2	28.8	0.895	3.47	1.36	0.36	7
4	13 USG 0-6	47.0	22.6	2.68	100	30.3	30.1	0.756	1.15	100	30.7	30.6	0.774	2.77	1.71	98.0	28.5	28.5	0.792	4.38	2.27	0.27	8
5	1 USG 0-6	45.1	22.4	2.70	69.0	34.7	34.4	1.37	0.25	72.0	32.6	32.3	1.23	1.47	0.41	97.9	35.5	34.0	1.32	2.51	0.40	0.55	4½
6	16 USG 0-6	45.3	19.8	2.68	85.8	38.7	38.3	1.21	0.39	87.3	40.0	38.2	1.23	1.60	0.54	86.0	37.4	36.6	1.16	2.68	0.57	0.21	3
7	2 USG 9-15	42.8	23.0	2.72	96.2	28.2	27.9	0.794	0.49	100	27.1	26.6	0.719	1.79	0.73	93.0	27.7	27.2	0.804	3.61	1.50	0.25	11½
8	4 USG 9-15	44.7	22.3	2.66	82.0	22.9	22.6	0.740	1.48	83	22.4	22.2	0.733	2.26	1.20	76.0	23.4	23.1	0.822	3.11	100	0.55	12
9	11 USG 0-6	43.2	22.3	2.73	52.5	31.9	31.9	1.66	0.15	59.8	31.6	30.9	1.44	1.45	0.39	53.2	31.1	30.5	1.59	2.65	0.54	0.06	7
11	4 USG 0-6	46.2	20.4	2.65	89.0	33.1	32.9	.982	0.95	93.0	31.5	30.9	0.896	2.21	1.15	93.0	31.6	29.6	.897	3.26	1.15	0.15	15
13	11 USG 9-15	40.0	21.0	2.72	90.0	35.7	34.7	1.08	0.15	82.6	34.8	33.3	1.15	1.43	0.37	89.5	34.5	33.1	1.05	2.56	0.44	0.21	3
14	12 USG 0-6	42.1	19.8	2.70	97.2	28.9	28.3	0.804	1.16	88.5	23.0	24.4	0.763	2.12	1.06	96.5	28.5	27.5	0.793	3.67	1.56	0.53	5
15	9 USG 0-6	41.7	20.0	2.65	89.2	23.4	23.0	0.692	1.17	93.8	21.7	21.2	0.610	2.70	1.64	90.5	20.6	20.3	0.602	3.91	1.80	0.50	11
16	14 USG 0-6	42.9	19.2	2.65	85.0	30.8	30.3	0.974	1.15	83.3	31.9	31.4	0.960	2.14	1.08	93.6	31.9	31.0	0.924	3.63	1.52	0.55	4½
17	3 USG 0-6	42.0	19.7	2.66	94	25.2	25.2	0.700	1.21	93	27.2	24.6	0.772	2.66	1.60	96	25.4	--	0.700	3.57	1.46	0.57	8
18	3 USG 9-15	39.7	20.1	2.67	98.0	21.0	--	0.570	1.34	58	24.9	--	1.16	2.71	1.65	85	22.9	--	0.717	3.74	1.63	0.66	6
19	2 USG 0-6	40.4	19.7	2.68	92.5	25.8	25.7	0.756	0.59	97.3	25.2	24.7	0.693	2.48	1.42	89.0	24.8	24.1	0.750	4.25	2.14	0.25	15

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED -- Saskatoon

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED: Degree Saturation %	UNCONFINED: % Moisture Start	UNCONFINED: % Moisture End	UNCONFINED: Void Ratio Start	UNCONFINED: Deviator Str. Kgm/cm²	15 p.s.i. LATERAL PRESSURE: Degree Saturation %	15 p.s.i.: % Moisture Start	15 p.s.i.: % Moisture End	15 p.s.i.: Void Ratio Start	15 p.s.i.: Max. Major Principal Stress Kgm/cm²	15 p.s.i.: Deviator Stress Kgm/cm²	30 p.s.i. LATERAL PRESSURE: Degree Saturation %	30 p.s.i.: Percent Moisture Start	30 p.s.i.: Percent Moisture End	30 p.s.i.: Void Ratio Start	30 p.s.i.: Max. Major Principal Stress Kgm/cm²	30 p.s.i.: Deviator Stress Kgm/cm²	'C' Cohesion Kgm/cm²	Angle of Internal Friction
20	5 USG 0-6	41.8	18.5	2.63	77.3	25.0	24.4	0.862	1.05	81.7	25.8	25.5	0.852	2.23	1.17	82.0	25.9	25.6	0.850	3.67	1.56	0.45	8
21	1 USG 6-12	39.5	19.0	2.73	100	24.6	24.2	0.643	1.74	100	24.7	24.4	0.642	2.20	1.14	100	26.0	26.6	0.673	3.66	1.55	1.81	7
22	20 USG 9-15	38.9	18.7	2.71	70.0	22.3	21.9	0.856	1.37	69.4	22.5	22.0	0.859	2.33	1.27	65.4	22.3	21.9	0.925	3.30	1.19	--	--
23	6 USG 0-6	41.1	17.2	2.72	85.0	26.0	25.7	0.819	0.58	83.7	26.2	25.4	0.839	2.80	1.74	88.0	25.9	25.4	0.788	4.11	2.00	0.23	21
24	8 USG 0-6	40.0	16.9	2.63	91.0	26.7	26.2	0.771	0.74	88	29.6	29.1	0.880	2.06	1.00	92.0	28.2	27.7	0.810	3.32	1.21	0.37	7
27	14 USG 9-15	38.8	15.0	2.62	92	29.6	29.4	0.846	0.54	94	26.7	26.5	0.745	2.40	1.34	92.5	31.6	30.1	0.896	3.15	1.04	0.25	15
28	12 USG 9-15	34.8	13.6	2.69	100	29.8	29.6	1.09	1.58	89.5	30.5	33.3	1.22	2.11	1.05	92.3	27.6	27.3	1.08	3.86	1.75	0.78	2
29	5 USG 9-15	28.8	11.5	2.69	88.0	29.0	28.6	0.886	0.61	87.3	27.9	27.8	0.858	1.73	0.67	88.3	28.0	27.7	0.849	2.96	0.85	0.30	4

Consolidation Test Data Summary

Soils Tested Saskatoon

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing →	10 USG 0-6	0.697	123	2.68	93.4	24.3	24.1	26.46		0.035	0.070	0.141	0.282	0.563	1.13	2.25			
										26.63	26.49	26.36	26.13	25.69	25.08	24.1			
	15 USG 0-6	0.641	128	2.71	100	23.8	22.4	25.8		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										26.20	26.16	25.96	25.61	25.12	24.32	22.4			
	(6-12) 10 USG 9-15	0.883	118	2.72	94.8	30.8	24.8	34.4		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										34.70	34.65	34.17	33.54	32.38	30.51	27.92	24.8		
	(12-18) 9 USG 9-15	0.729	123	2.71	93.0	25.0	21.7	27.0		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										27.30	27.29	27.04	26.70	26.24	25.32	23.94	21.7		
Plasticity	(12-18) 15 USG 9-15	1.05	114	2.74	96.0	36.9	29.4	40.4		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										40.52	40.50	39.92	39.10	37.68	35.75	33.04	29.4		
	(6-12) 11 USG 9-15	0.998	112	2.72	87.0	31.6	29.7	35.2		0.040	0.069	0.155	0.302	0.591	1.17	2.34			
										35.09	35.07	34.78	34.38	33.62	31.98	29.7			
	17 USG 0-6	0.731	136	2.67	97.0	26.4	25.1	28.0		0.040	0.069	0.155	0.302	0.591	1.17	2.34			
										28.05	28.04	27.98	27.79	27.41	26.54	25.1			
Decreasing	(12-18) 17 USG 9-15	1.07	101	2.70	59.2	23.4	28.7	33.7		0.040	0.069	0.155	0.302	0.591	1.17	2.34			
										33.85	33.82	33.66	33.35	32.71	31.66	28.7			
	(12-18) 16 USG 9-15	0.850	107	2.70	55.0	17.4	23.5	31.3		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										31.30	31.27	30.93	30.47	29.70	28.50	26.44	23.5		
	13 USG 0-6	0.809	119	2.70	93.1	27.9	23.8	30.6		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										30.81	30.73	30.35	29.85	28.99	27.91	26.28	23.8		
↓	(12-18) 13 USG 9-15	0.973	113	2.73	83.6	29.8	29.2	35.2		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										35.42	35.37	35.05	34.54	33.61	32.14	29.2			

Consolidation Test Data Summary

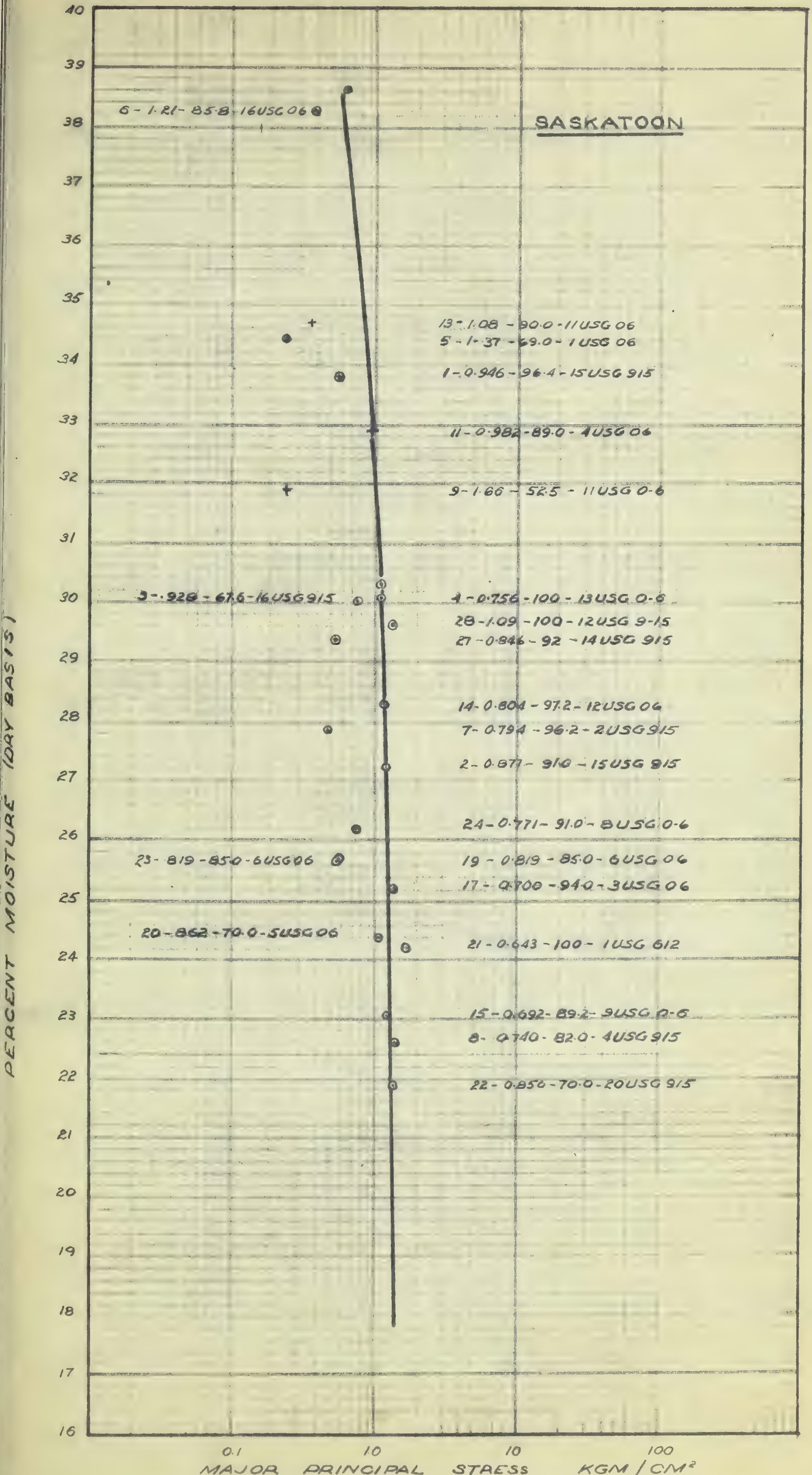
Soils Tested ....

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing →	4 USG 9-15	0.652	123	2.66	91.1	22.3	24.0	25.6		0.040	0.069	0.155	0.302	0.591	1.17	2.34			
										26.30	26.29	26.23	26.07	25.73	25.12	24.0			
	1 USG 0-6	0.827	120	2.70	96.8	29.6	29.3	33.9		0.039	0.068	0.154	0.296	0.583	1.16	2.30	4.30	8.58	0.039
										33.22	33.20	32.75	32.33	31.31	30.11	28.58	26.84	24.55	29.3
	16 USG 0-6	0.953	114	2.68	94.2	33.5	28.3	35.1		0.043	0.072	0.158	0.300	0.586	1.16	2.29			
										35.02	34.99	34.58	34.10	32.84	31.18	28.3			
	4 USG 0-6	0.827	112	2.65	86.5	27.0	27.0	30.2		0.043	0.074	0.162	0.309	0.604	1.19	2.37			
										30.40	30.38	30.18	29.86	29.38	28.58	27.0			
Plasticity	2 USG 9-15	0.915	119	2.70	100	34.0	26.6	34.0		0.041	0.070	0.157	0.303	0.595	1.18	2.35			
										33.56	33.50	33.29	32.85	31.55	29.54	26.6			
	(6-12) 14 USG 9-15	0.797	120	2.62	93.5	28.4	23.8	31.0		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										31.01	30.94	30.71	30.20	29.45	28.15	26.35	23.8		
	14 USG 0-6	0.864	119	2.66	82.8	26.9	28.2	31.4		0.039	0.068	0.154	0.296	0.583	1.16	2.30			
										31.42	31.40	31.23	31.00	30.56	29.69	28.2			
Decreasing	(12-18) 5 USG 9-15	0.854	117	2.63	93.0	29.7	26.4	32.0		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										31.92	31.85	31.42	30.85	29.90	28.55	26.4			
	(12-18) 6 USG 9-15	1.03	113	2.71	92.0	35.2	28.8	37.0		0.039	0.068	0.154	0.296	0.583	1.16	2.30			
										36.31	36.30	36.25	35.68	34.06	31.91	28.8			
←	3 USG 0-6	0.701	122	2.69	75.5	23.6	24.6	27.1		0.086	0.158	0.300	0.725	1.44	2.91	5.81	0.086		
										27.48	27.42	26.99	26.16	25.33	23.96	22.08	24.6		

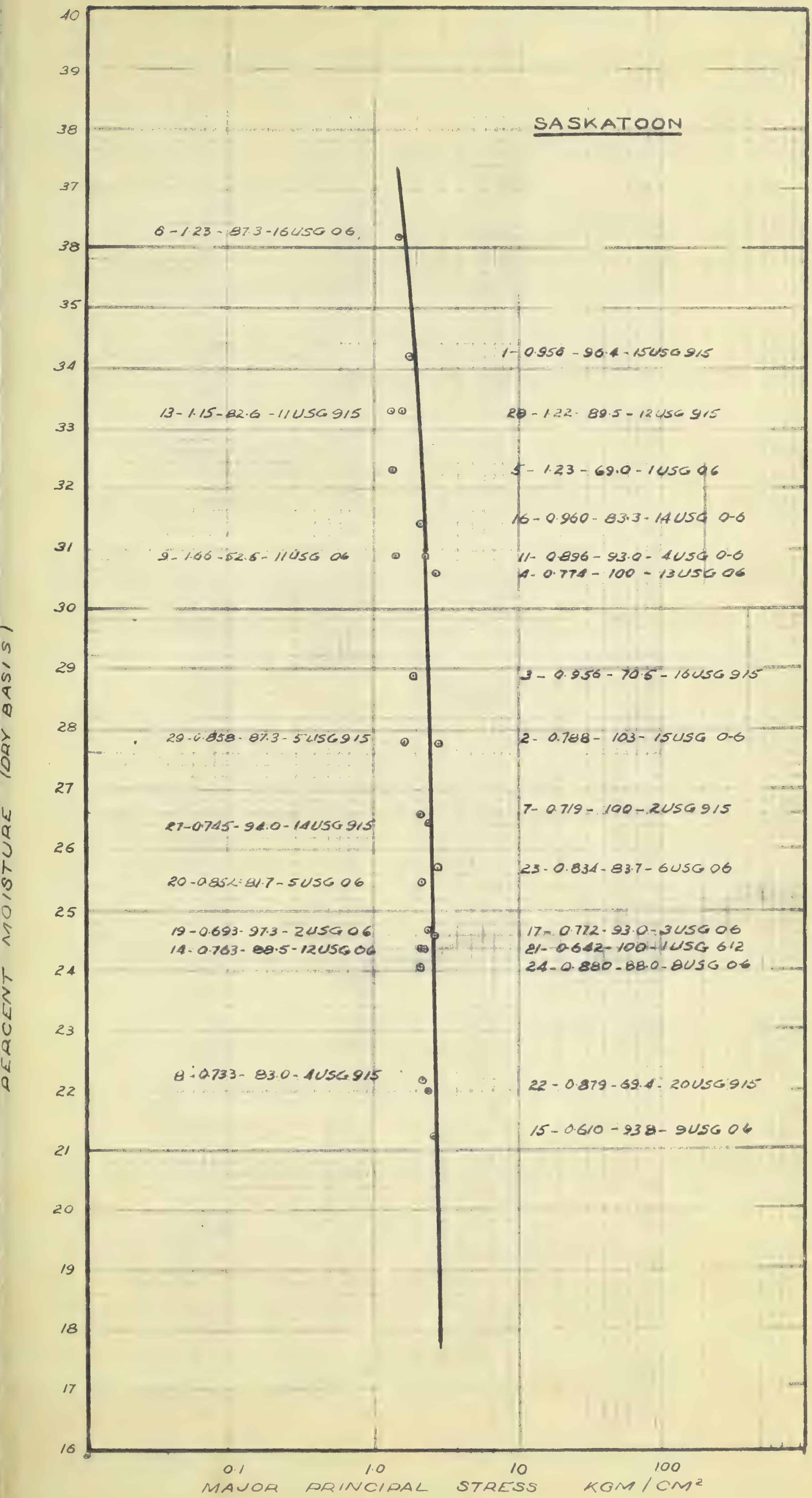
Consolidation Test Data Summary

Soils Tested Saskatoon

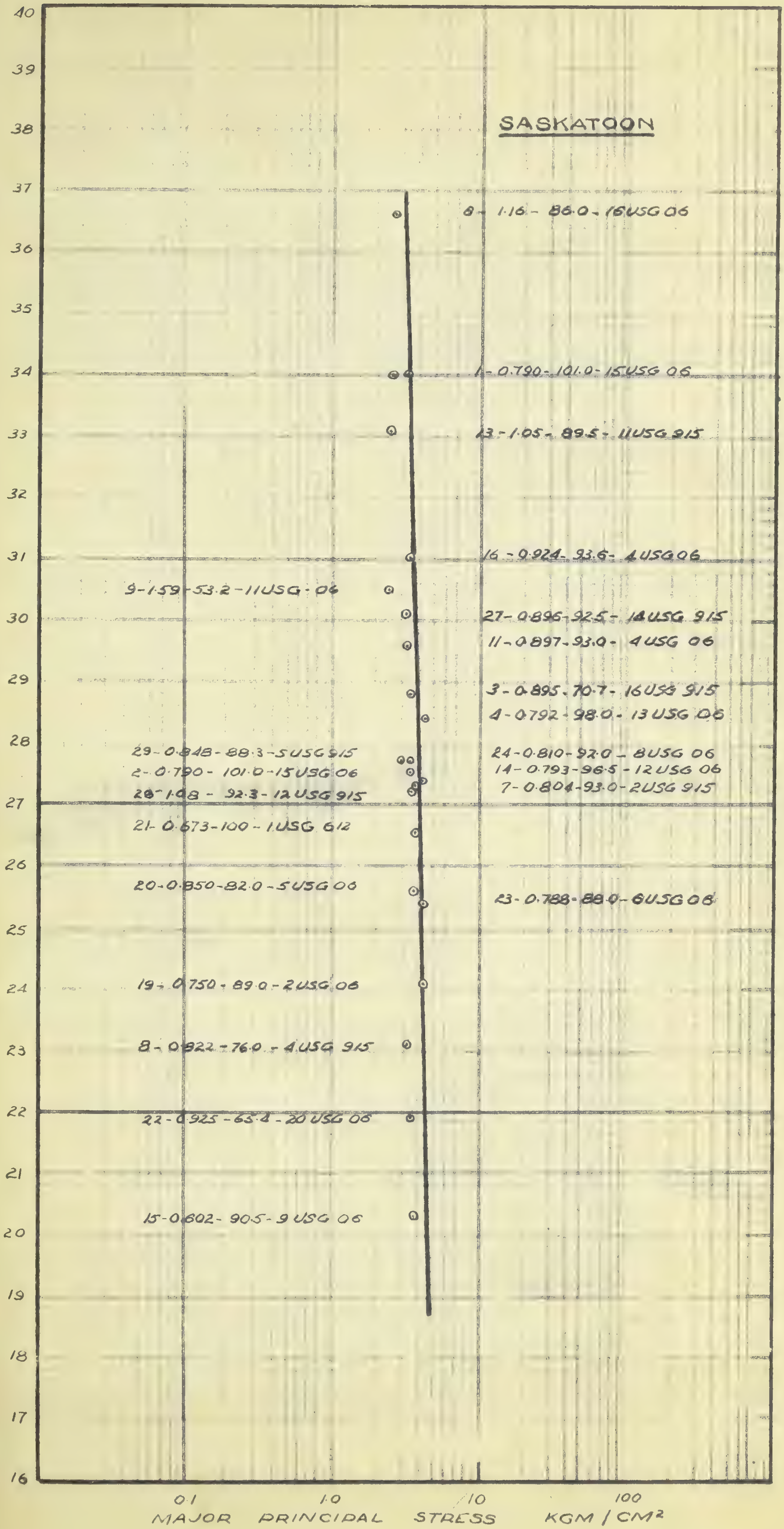
	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing ↑	12 USG 0-6	0.964	109	2.70	73.7	26.3	29.6	32.9		0.043	0.072	0.158	0.300	0.586	1.16	2.29			
										32.86	32.87	32.70	32.43	31.89	31.02	29.6			
	(12-18) 3 USG 9-15	0.721	120	2.67	87.0	23.4	25.3	27.9		0.035	0.070	0.141	0.281	0.563	1.13	2.25	0.035		
										27.92	27.83	27.58	27.16	26.57	25.29	23.24	25.3		
	5 USG 0-6	0.787	111	2.65	65.3	19.4	22.2	65.3		0.041	0.070	0.158	0.306	0.600	1.19	2.37			
										25.36	25.34	25.22	25.06	24.62	23.88	22.2			
	2 USG 0-6	0.814	119	2.70	93.6	28.2	25.9	30.1		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										30.38	30.35	30.08	29.66	28.92	27.74	25.9			
Plasticity	1 USG 6-12	0.941	117	2.74	100	34.7	28.9	34.7		0.043	0.072	0.158	0.300	0.586	1.16	2.29			
										34.68	34.65	34.39	33.83	32.87	31.12	28.9			
	6 USG 0-6	0.709	122	2.72	86	22.4	22.6	24.6		0.035	0.070	0.160	0.305	0.595	1.18	2.33			
										25.52	24.63	24.50	24.36	24.08	23.53	22.6			
	9 USG 0-6	0.659	123	2.65	93.4	23.2	23.4	25.0		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										25.52	25.48	25.38	25.19	24.88	24.35	23.4			
	(12-18) 18 USG 9-15	1.20	98.5	2.63	69.7	31.1	33.7	39.6		0.041	0.070	0.158	0.306	0.600	1.19	2.37			
										39.72	39.69	39.57	39.25	38.62	37.16	33.7			
Decreasing ↓	8 USG 9-15	0.787	107	2.63	54.6	16.4	20.1	28.9		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										29.35	29.30	29.10	28.81	28.24	26.53	23.48	20.1		
	(6-12) 12 USG 9-15	0.788	124	2.69	98.8	27.9	23.8	30.6		0.035	0.070	0.141	0.281	0.563	1.13	2.25			
										31.82	31.78	31.52	31.24	30.70	29.76	28.2			



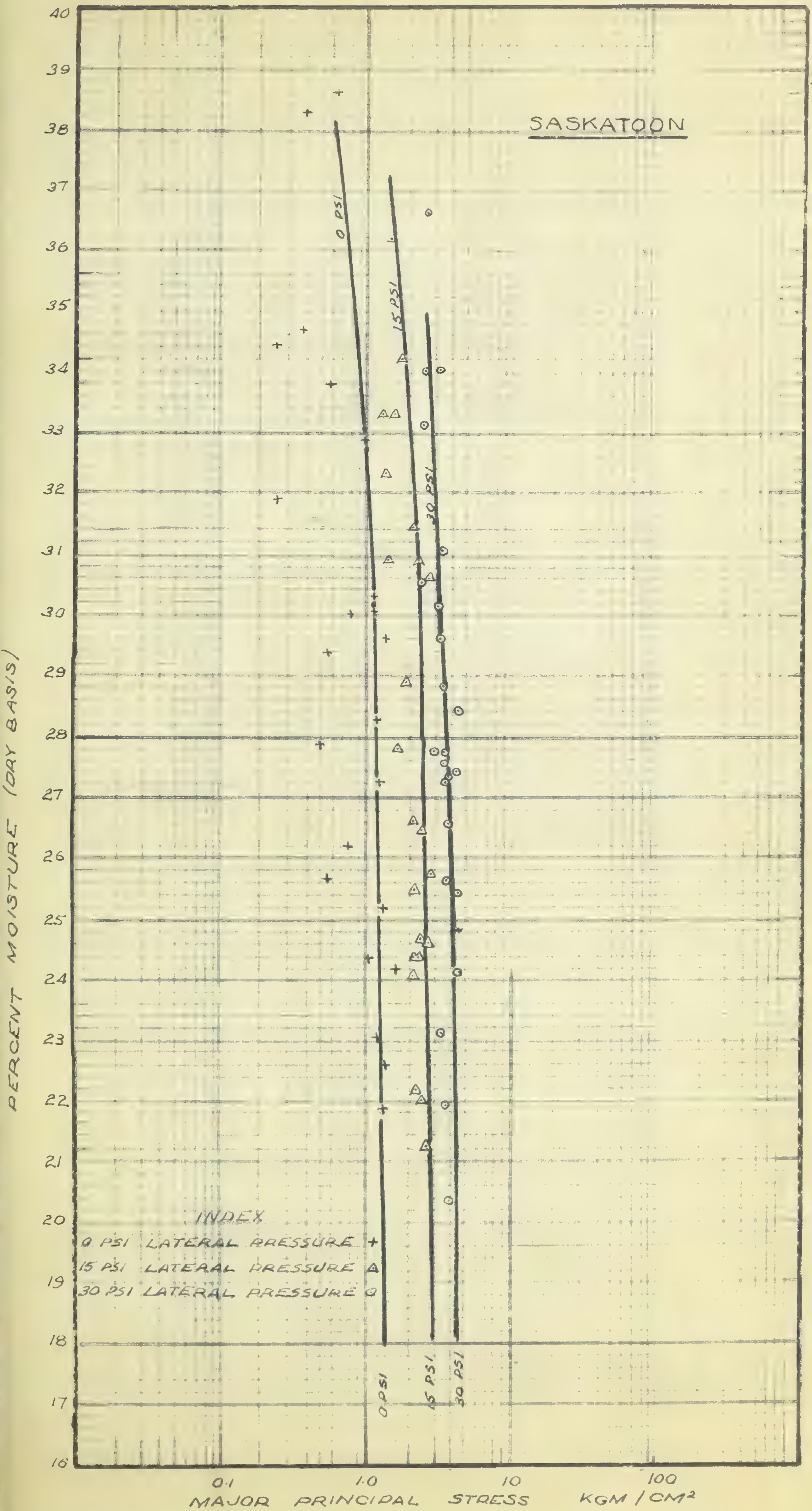
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 0 - LATERAL PRESSURE



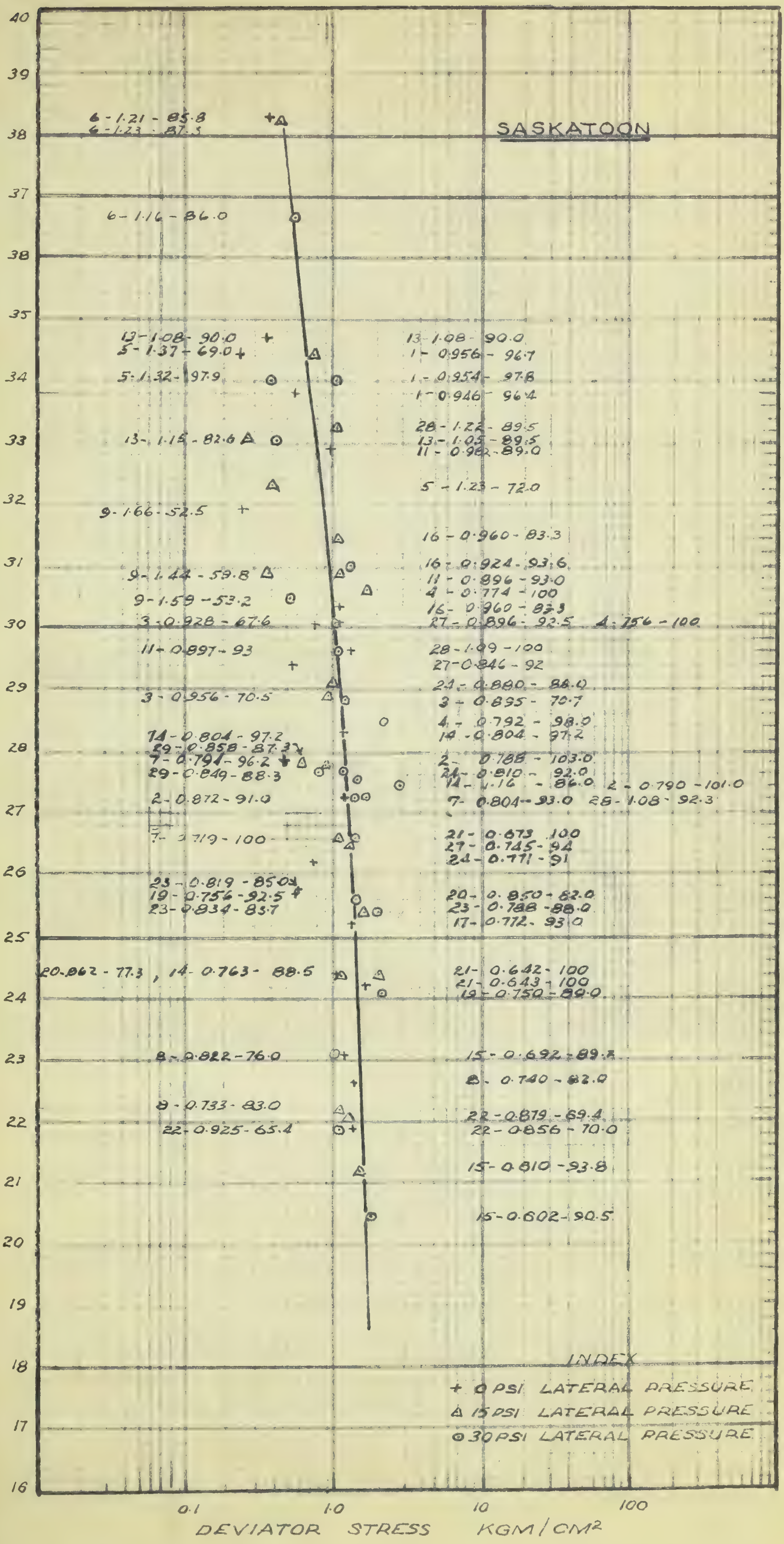
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 15 PSI LATERAL PRESSURE



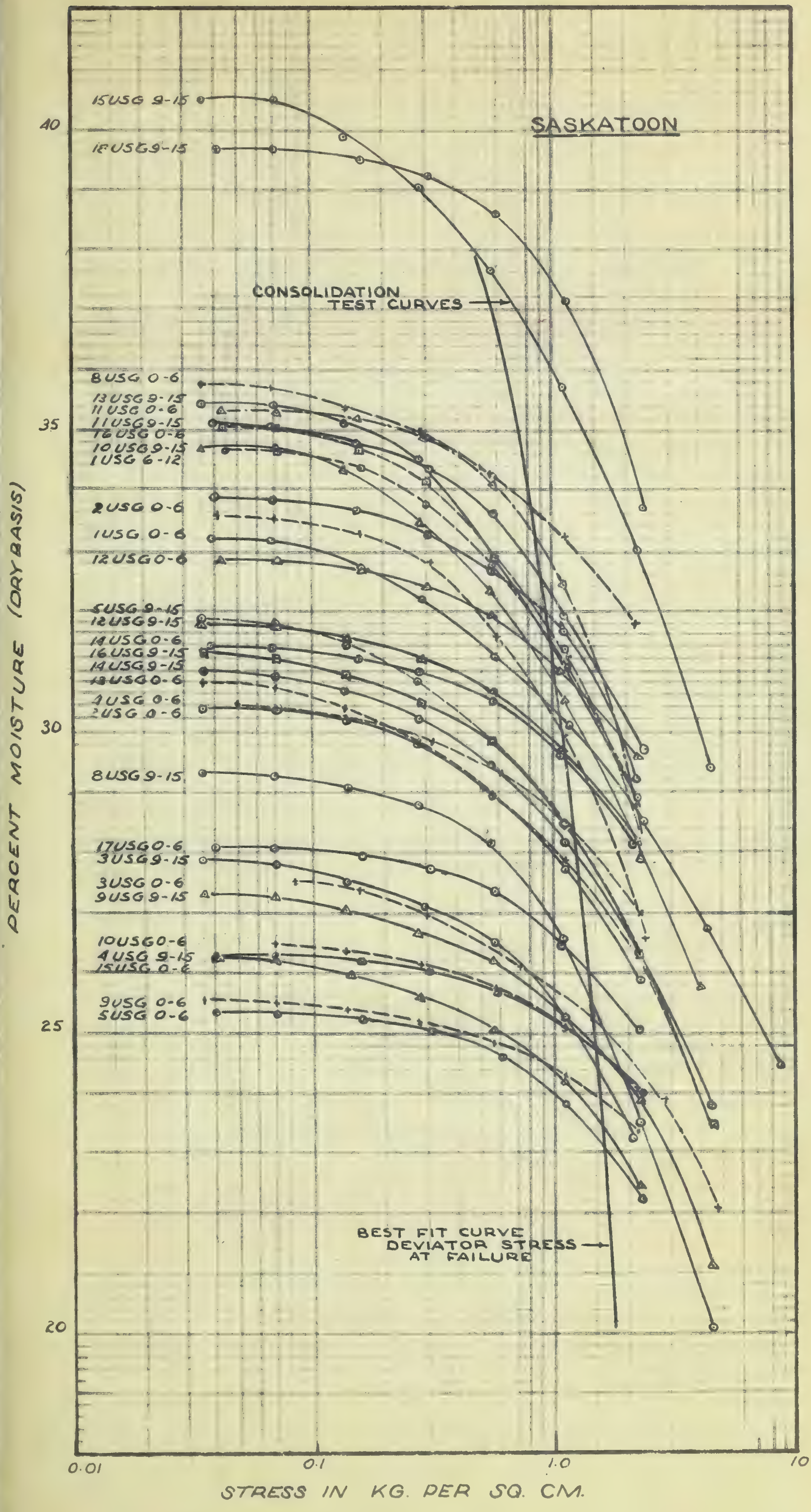
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 30 PSI LATERAL PRESSURE



PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS

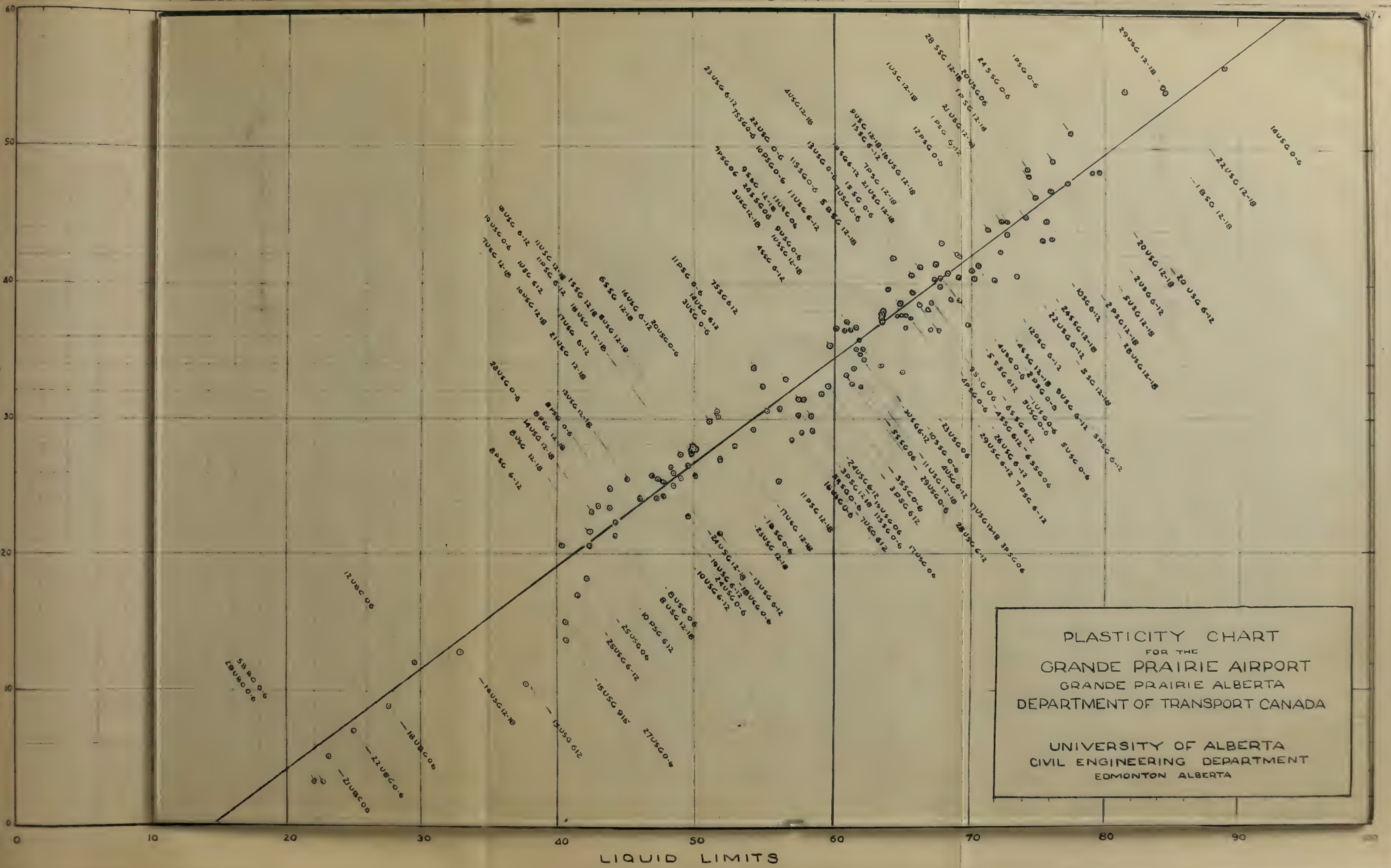


PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS. DEVIATOR STRESS



CONSOLIDATION AND TRIAXIAL TEST MOISTURE - STRESS RELATIONS

GRAND PRAIRIE


PLASTICITY CHART
FOR THE
GRANDE PRAIRIE AIRPORT
GRANDE PRAIRIE ALBERTA
DEPARTMENT OF TRANSPORT CANADA
UNIVERSITY OF ALBERTA
CIVIL ENGINEERING DEPARTMENT
EDMONTON ALBERTA
0
10
20
30
40
50
60
70
80
90
LIQUID LIMITS
0
10
20
30
40
50
60

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED					15 p.s.i. LATERAL PRESSURE						30 p.s.i. LATERAL PRESSURE						'C' Cohesion Kgm/cm^2	Angle of Internal Friction
					Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Deviator Str. Kgm/cm^2	Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm^2	Deviator Stress Kgm/cm^2	Degree Saturation %	Percent Moisture Start	Percent Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm^2	Deviator Stress Kgm/cm^2		
1	14 USG 0-6	89.3	56.1	2.72	70.8	18.93	18.58	0.728	2.88	6.66	19.11	18.97	0.781	5.45	4.39	65.0	23.23	23.17	0.973	5.46	3.35	0.93	25
2	29 USG 9-15	84.5	54.8	2.66	49.3	19.96	19.30	1.08	2.48	57.7	21.84	21.43	1.01	4.87	3.81	62.8	21.47	20.57	0.911	7.57	5.46	0.88	25
3	20 USG 9-15	79.0	48.1	2.72	57.1	21.97	21.31	1.04	0.50	74.0	25.22	21.31	0.922	7.54	6.48	80.2	24.51	24.16	0.825	0.800	5.89	--	--
4	2 USG 9-15	77.3	47.3	2.69	103	31.6	--	0.923	0.809	100	30.8	--	0.902	1.95	0.89	102	30.6	--	0.878	3.26	1.15	0.38	4
6	1 USG 10-16	72.4	45.9	2.70	96.8	31.0	--	0.859	1.37	96.7	31.3	--	0.870	2.54	1.48	97.5	31.2	--	0.835	3.84	1.73	0.62	5
8	9 USG 9-15	69.5	42.5	2.70	73.5	26.19	25.98	0.973	2.03	87.6	26.09	25.77	0.821	3.64	2.58	89.5	25.28	25.07	0.779	5.76	3.65	0.73	17
9	22 USG 9-15	72.5	42.2	2.72	98.8	32.6	--	0.883	0.857	95.0	31.9	--	0.896	2.13	1.07	87.8	32.0	--	0.962	3.30	1.19	0.41	5
10	4 USG 0-6	69.4	41.7	2.68	84.2	24.67	24.39	0.774	1.99	83.7	23.14	22.98	0.728	3.72	2.66	82.3	24.24	23.96	0.678	5.38	3.27	0.56	34.5
11	4 USG 9-15	70.2	40.4	2.66	69.3	21.59	21.32	0.850	1.52	67.0	23.97	23.72	0.977	5.96	4.90	65.7	22.31	22.11	0.925	11.64	9.53	0.38	41
12	13 USG 0-6	64.2	41.8	2.62	74.5	20.76	20.57	0.731	1.98	64.3	19.49	19.19	0.894	3.82	2.76	76.5	20.39	20.04	0.700	6.38	4.22	0.72	20.5
13	1 USG 0-6	65.6	40.8	2.68	99.0	25.9	20.4	0.698	1.98	101	26.2	27.1	0.698	2.90	1.84	96.0	24.6	21.4	0.667	4.35	2.24	0.85	13
14	21 USG 9-15	67.7	39.8	2.68	63.6	21.87	20.99	0.910	0.50	68.8	24.56	24.07	0.967	6.46	5.40	70.7	21.48	21.23	0.822	8.38	6.27	1.70	16
15	5 USG 9-15	69.2	38.8	2.69	81.1	22.56	22.43	0.736	2.45	72.0	20.26	20.22	0.746	4.43	3.37	72.0	21.12	21.06	0.777	6.91	4.80	0.86	21
16	7 USG 0-6	65.5	39.2	2.71	84.0	16.5	16.5	1.09	1.80	47.0	17.3	16.8	1.06	4.87	3.81	50.3	14.4	14.9	0.950	5.90	3.79	0.54	29
18	22 USG 0-6	65.0	37.6	2.55	90.0	30.2	--	1.00	0.701	99.0	31.0	--	0.816	1.56	0.50	90.0	31.2	--	0.822	3.00	0.89	0.35	2
19	5 USG 0-6	69.7	36.9	2.63	86.2	26.1	26.0	0.802	1.98	83.9	25.64	25.48	0.746	4.43	3.37	72.0	21.12	21.06	0.777	6.91	4.80	--	--
20	10 USG 9-15	--	--	2.68	75.1	16.3	--	0.554	1.51	75.1	17.9	--	0.628	3.92	2.86	66.7	17.6	--	0.702	4.82	2.71	--	--

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED-- Grand Prairie

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED					15 p.s.i. LATERAL PRESSURE						30 p.s.i. LATERAL PRESSURE						'C' Cohesion Kgm/cm²	Angle of Internal Friction
					Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Deviator Str. Kgm/cm²	Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm²	Deviator Stress Kgm/cm²	Degree Saturation %	Percent Moisture Start	Percent Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm²	Deviator Stress Kgm/cm²		
22	3 USG 9-15	62.0	35.0	2.67	78.0	25.67	25.40	0.922	2.06	73.0	24.71	24.34	0.905	4.63	3.57	78.3	25.66	25.55	0.878	6.11	4.00	0.68	26.5
23	9 USG 0-6	61.8	35.6	2.69	92.3	25.49	25.19	0.745	1.93	82.8	25.38	25.22	0.825	2.52	1.46	88.9	25.67	18.51	0.776	4.67	2.56	0.90	6.5
24	27 USG 9-15	61.9	34.5	2.71	77.1	28.54	21.11	0.981	0.50	68.6	25.23	24.82	0.975	4.20	3.14	72.9	26.55	26.11	0.966	5.03	2.92	--	--
24	27 USG 9-15	61.9	34.5	2.71	63.2	22.2	21.9	0.924	0.40	68.9	23.1	22.8	0.877	2.99	1.93	73.7	23.1	22.8	0.821	4.05	1.94	0.15	24
25	23 USG 0-6	65.0	33.4	2.50	85.1	20.8	20.3	0.648	1.91	93.0	21.6	21.6	0.614	4.20	3.14	86.0	21.4	21.4	0.641	6.10	3.99	0.63	22
26	29 USG 0-6	60.8	33.1	2.68	68.5	16.7	16.4	0.651	1.00	61.9	16.49	16.28	0.715	5.32	4.26	53.1	17.15	16.95	0.865	5.91	3.80	0.32	36
27	24 USG 9-15	57.9	31.4	2.70	56.3	15.69	14.83	0.752	1.01	55.4	15.22	14.98	0.741	8.30	7.24	47.3	13.59	15.81	0.774	6.94	4.83	0.28	43
29	18 USG 0-6	57.5	31.0	2.70	97.2	25.8	--	0.724	1.39	95.3	25.9	--	0.742	2.73	1.66	100	27.3	--	0.742	3.80	1.79	0.62	6
30	17 USG 0-6	58.4	30.2	2.68	63.8	17.34	17.70	0.749	1.48	60.5	17.58	17.65	0.778	5.01	3.95	65.3	17.62	17.07	0.723	6.84	4.73	0.41	32.5
31	20 USG 0-6	51.6	30.3	2.66	64.7	18.09	17.72	0.747	2.03	61.2	18.84	18.55	0.822	6.76	5.70	70.7	19.91	19.77	0.754	6.43	4.32	0.63	37
32	3 USG 0-6	55.2	30.6	2.69	88.9	22.2	21.7	0.672	2.91	84.0	22.7	22.6	0.812	4.38	3.32	85.8	21.4	21.15	0.669	6.49	4.38	1.15	14
33	18 USG 0-6	57.5	31.0	2.70	61.5	17.72	17.21	0.778	1.00	61.6	16.96	16.44	0.743	6.15	5.09	69.7	16.65	16.35	0.645	9.33	7.22	0.90	29
34	16 USG 0-6	57.0	28.4	2.64	81.0	23.26	22.92	0.974	0.97	62.6	20.89	19.92	0.881	4.15	3.09	67.3	23.65	23.46	0.925	6.47	4.36	0.34	26
35	16 USG 6-12	51.7	30.1	2.69	64.0	20.8	20.5	0.866	1.52	65.0	20.9	--	0.863	4.63	3.57	--	--	--	--	---	--	--	--
36	17 USG 9-15	52.8	27.9	2.69	49.5	16.34	15.93	0.861	0.49	48.4	16.00	15.95	0.890	5.05	3.99	54.0	17.24	17.22	0.862	6.92	4.81	1.29	16
37	8 USG 9-15	42.5	21.7	2.67	65.0	19.56	19.18	0.811	0.99	73.5	21.05	20.87	0.775	4.24	3.18	68.6	20.27	20.18	0.802	6.06	3.95	1.02	14
38	19 USG 9-15	49.8	27.6	2.67	79.2	21.80	21.38	0.729	1.00	68.1	20.41	20.21	0.802	2.53	1.47	76.1	21.38	21.18	0.751	4.82	2.71	0.40	17

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED-- Grand Prairie

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED Degree Saturation %	UNCONFINED % Moisture Start	UNCONFINED % Moisture End	UNCONFINED Void Ratio Start	UNCONFINED Deviator Str., Kgm/cm2	15 p.s.i. LATERAL PRESSURE Degree Saturation %	15 p.s.i. % Moisture Start	15 p.s.i. % Moisture End	15 p.s.i. Void Ratio Start	15 p.s.i. Max. Major Principal Stress Kgm/cm2	15 p.s.i. Deviator Stress2 Kgm/cm	30 p.s.i. LATERAL PRESSURE Degree Saturation %	30 p.s.i. Percent Moisture Start	30 p.s.i. Percent Moisture End	30 p.s.i. Void Ratio Start	30 p.s.i. Max. Major Principal Stress Kgm/cm2	30 p.s.i. Deviator Stress Kgm/cm2	'C' Cohesion Kgm/cm2	Angle of Internal Friction
39	7 USG 9-15	46.9	25.7	2.67	57.8	16.3	--	0.752	3.25	--	--	--	--	--	--	58.0	14.8	--	0.714	6.65	4.54	1.30	13
40	13 USG 9-15	45.1	25.6	2.65	74.4	18.18	17.74	0.650	0.50	62.7	17.39	17.15	0.719	6.12	5.06	65.7	17.89	17.30	0.724	12.71	10.60	0.2	45
41	15 USG 0-6	49.6	22.8	2.65	72.3	24.32	23.53	0.918	1.02	71.6	23.64	23.41	0.896	4.32	3.26	64.7	23.75	22.79	0.996	8.67	6.56	0.27	35
42	10 USG 0-6	46.0	21.9	2.68	92.5	21.2	--	0.618	1.58	97.7	24.5	--	0.667	3.30	2.24	98.8	24.2	--	0.658	4.73	2.62	0.62	14
43	28 USG 0-6	43.0	23.7	2.69	76.5	21.26	20.98	0.750	1.48	70.1	21.37	21.14	0.818	4.02	2.96	68.2	21.54	21.37	0.849	5.63	3.52	0.54	19
44	8 USG 0-6	44.2	22.5	2.79	72.8	18.41	18.23	0.702	2.67	84.6	19.90	19.76	0.653	4.33	5.39	80.3	22.6	--	0.803	6.46	4.35	0.88	22.5
45	27 USG 0-6	40.7	14.9	2.65	86.2	28.41	21.2	0.814	3.02	63.8	20.2	--	0.631	5.29	4.23	80.3	22.6	--	0.803	6.46	4.35	--	--
45	27 USG 0-6	40.7	14.9	2.65	65.5	20.40	19.90	0.815	1.00	68.2	24.24	23.61	0.933	6.27	5.21	67.5	20.72	20.31	0.805	9.62	7.51	0.96	30.5
46	15 USG 9-15	40.6	13.8	2.66	60.8	22.52	21.41	0.985	1.00	68.0	22.15	21.57	0.868	3.65	2.59	73.3	25.08	24.38	0.909	6.89	4.78	--	--
	26 USG 0-6	No Limits		2.67	86.6	24.8	--	0.770	2.40	91.8	25.0	--	0.734	3.58	2.52	89.3	24.4	--	0.733	4.27	2.16	1.12	4
	26 USG 0-6	No Limits		2.67	52.6	20.10	19.12	1.02	0.50	54.9	20.30	20.10	0.986	2.46	3.52	58.3	20.46	19.97	0.940	6.88	4.77	--	--
	26 USG 9-15	No Limits		2.67	58.1	20.96	20.66	1.05	--	51.9	22.51	21.92	0.955	2.88	1.82	52.0	23.03	22.48	1.19	5.72	3.61	0.72	26

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED -- Grand Prairie

Consolidation Test Data Summary

Soils Tested Grand Prairie

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing →										0.034	0.070	0.144	0.295	0.590	1.18	2.36	4.73		
	20 USG 9-15	0.609	120	2.70	62.3	14.0	19.3	--		26.43	26.39	26.21	25.7	24.9	23.5	21.9	19.3		
	(12-18)									0.040	0.070	0.157	0.306	0.600	1.19	2.37	4.43		
	5 USG 9-15	0.762	118	2.67	83.7	23.6	30.3	33.3		39.7	39.6	39.3	38.3	37.1	35.5	33.1	30.3		
	(12-18)									0.035	0.072	0.146	0.296	0.599	1.20	2.39	4.78		
	28 USG 9-15	0.947	106	2.67	65.5	23.2	39.9	47.2		47.1	46.4	44.5	42.3	40.5	36.0	33.1	39.9		
										0.035	0.070	0.141	0.282	0.570	1.14	2.30	4.36		
	21 USG 0-6	0.868	110	2.71	64.4	20.5	30.6	35.1		49.6	42.7	42.3	41.2	39.2	36.8	33.7	30.6		
Plasticity										0.035	0.070	0.141	0.281	0.562	1.12	2.26	4.30	8.70	
	9 USG 9-15	0.762	122	2.70	95.4	26.8	30.0	34.9		42.2	41.2	41.1	40.2	39.0	37.6	35.6	33.2	30.0	
										0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
	16 USG 9-15	0.912	106	2.69	61.4	20.8	30.7	37.1		38.2	38.1	37.8	37.1	35.5	32.9	29.8	26.7		
										0.040	0.069	0.157	0.301	0.590	1.17	2.33	4.36		
	5 USG 0-6	0.737	117	2.67	86.9	24.5	30.5	33.9		38.1	38.0	37.6	36.9	35.7	34.7	32.8	30.5		
										0.035	0.073	0.141	0.288	0.570	1.14	2.30	4.36		
	13 USG 0-6	0.531	127	2.64	85.2	17.1	22.3	24.0	2 Samples	28.7	28.7	28.3	27.7	26.6	25.3	23.9	22.3		
Decreasing										0.041	0.070	0.158	0.306	0.600	1.19	2.37			
	13 USG 0-6	0.721	109	2.68	42.6	11.4	22.6	24.2		25.0	25.0	24.8	24.6	24.2	23.5	22.6			
										0.035	0.070	0.140	0.281	0.562	1.12	2.25	4.50		
	7 USG 0-6	0.762	120	2.68	94.4	27.0	26.1	30.6		32.2	32.1	32.0	31.7	30.9	29.0	28.4	26.1		
	(12-18)									0.040	0.069	0.156	0.301	0.590	1.17	2.33			
←	4 USG 9-15	0.750	115	2.66	86.5	21.5	30.2	38.5		38.4	38.4	38.0	37.0	35.4	33.2	30.2			

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing	26 USG 9-15	1.58	80.2	2.67	40.5	23.9	46.7	57.7		0.040	0.069	0.156	0.302	0.590	1.17	2.33			
										57.7	56.8	56.5	54.1	51.8	51.0	46.7			
	23 USG 0-6	0.871	109	2.58	81.0	27.3	32.5	35.1		0.035	0.070	0.141	0.281	0.562	1.13	2.25			
										35.7	35.7	35.6	35.2	34.5	32.9	30.5			
	9 USG 0-6	0.762	128	2.69	94.1	26.5	27.4	29.7		0.042	0.072	0.158	0.300	0.586	1.16	2.30	4.30		
										32.6	32.6	32.4	31.9	31.2	30.3	29.1	27.4		
	27 USG 9-15	0.809	112	2.66	72.8	22.1	28.0	34.9		0.048	0.073	0.158	0.306	0.600	1.19	2.37	4.43		
										34.9	34.9	34.8	34.5	33.9	32.9	30.6	28.0		
Plasticity	3 USG 9-15	0.850	115	2.67	85.3	26.9	31.3	32.6		0.035	0.070	0.141	0.288	0.570	1.14	2.30			
										37.1	35.7	36.4	34.9	34.4	33.4	31.3			
	29 USG 0-6	0.850	107	2.68	62.6	32.1	31.3	34.7		0.035	0.069	0.134	0.281	0.564	1.12	2.26			
										40.8	40.7	40.4	39.1	37.7	34.4	31.3			
	18 USG 0-6	0.897	101	2.70	42.9	14.3	25.0	27.6		0.036	0.073	0.149	0.302	0.611	1.22	2.44	4.88		
										33.0	32.9	32.3	31.5	30.6	29.0	27.1	25.0		
Decreasing	14 USG 9-15	0.850	112	2.73	67.3	20.8	30.0	32.9		0.036	0.071	0.142	0.289	0.581	1.16	2.34	4.43		
										38.4	38.1	37.9	36.9	34.8	34.1	32.1	30.0		
	3 USG 0-6	0.583	129	2.69	100	22.1	22.4	23.9		0.035	0.070	0.141	0.281	0.563	1.13	2.25	4.50		
										27.1	27.1	26.9	26.5	25.7	24.8	23.8	22.4		
	16 USG 0-6	0.814	112	2.64	67.8	21.1	30.7	33.3		0.039	0.068	0.154	0.296	0.583	1.15	2.29			
										36.5	36.5	36.3	35.7	34.8	33.3	30.7			
	20 USG 0-6	0.575	123	2.66	75.7	16.2	20.9	23.6		0.039	0.068	0.154	0.296	0.583	1.15	2.30	4.30		
										27.4	27.4	27.0	25.8	25.1	24.8	23.1	20.9		

Consolidation Test Data Summary

Soils Tested ..Grand Prairie.....

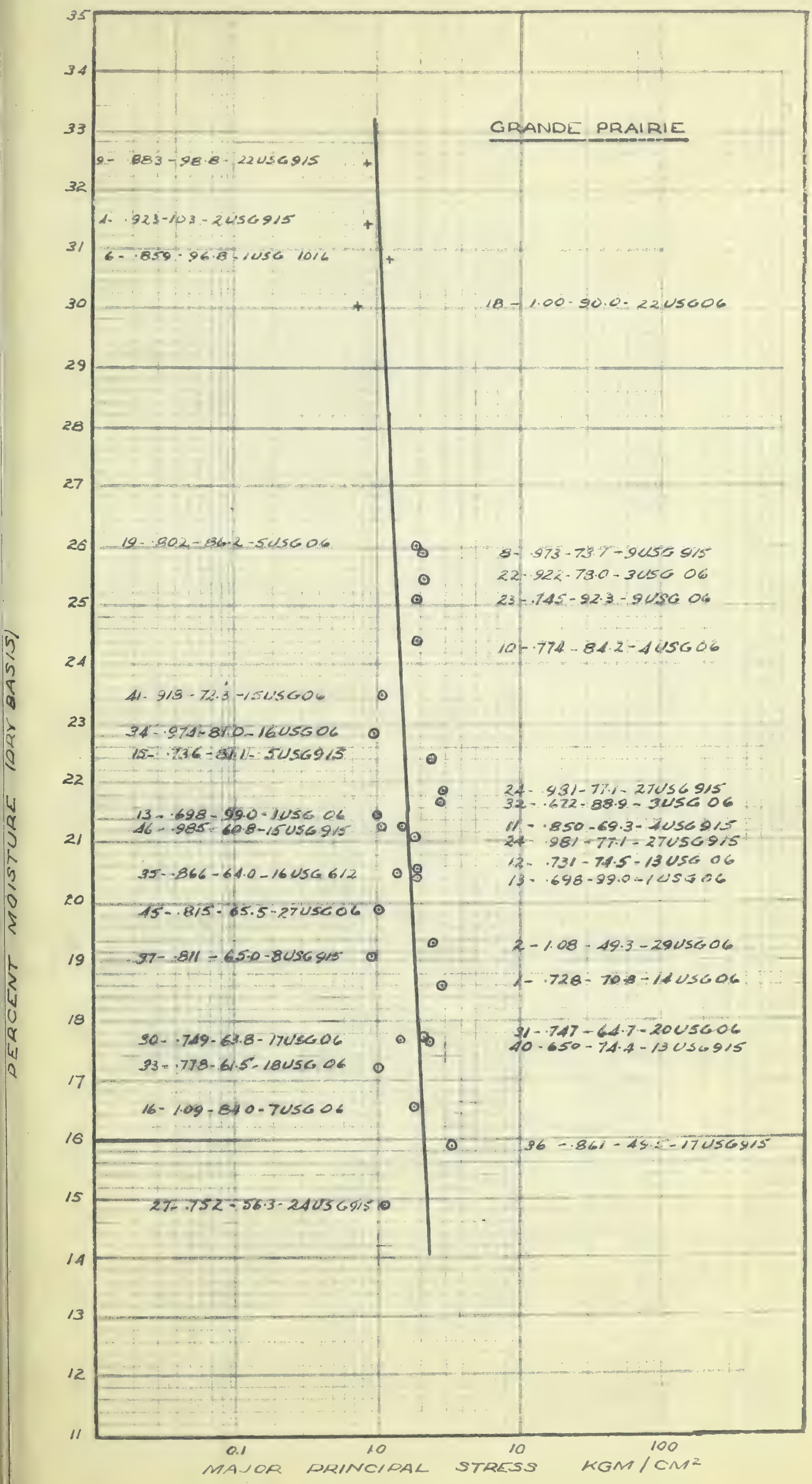
	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
↑	(6-12) 8 USG 9-15	0.609	123	2.68	78.6	177	21.1	24.3		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.50		
										26.0	26.0	25.8	25.2	24.4	23.5	22.5	21.1		
Increasing	18 USG 9-15	0.707	111	2.68	51.6	13.7	23.0	27.6		0.040	0.069	0.156	0.302	0.590	1.17	2.19			
										30.6	30.6	30.4	29.5	28.0	25.5	23.0			
	24 USG 9-15	0.633	120	2.67	68.4	16.1	28.2	31.6		0.040	0.069	0.156	0.302	0.592	1.17	2.32			
										32.7	32.7	32.5	32.1	31.4	30.2	28.2			
	(12-18) 21 USG 9-15	0.985	106	2.71	65.0	23.0	32.3	37.8		0.043	0.070	0.156	0.300	0.588	1.16	2.30			
										40.1	40.0	39.6	38.6	37.5	35.3	32.3			
Plasticity	(6-12) 19 USG 9-15	0.644	122	2.67	89.7	21.9	24.4	26.4		0.042	0.072	0.157	0.300	0.785	1.16	2.30			
										27.7	27.7	27.5	27.1	26.3	25.6	24.4			
	15 USG 0-6	0.805	113	2.65	76.9	23.5	28.5	31.6		0.039	0.068	0.154	0.397	0.583	1.15	2.29			
										33.8	33.7	33.6	33.0	32.4	30.9	28.5			
	(12-18) 13 USG 9-15	0.650	115	2.65	61.7	15.4	25.5	27.5		0.035	0.073	0.141	0.288	0.570	1.14	2.30			
										29.9	29.8	29.7	29.1	28.1	26.9	25.5			
	8 USG 0-6	0.805	123	2.77	93.0	26.8	29.0	30.5		0.044	0.073	0.160	0.305	0.595	1.18	2.41	4.36		
										35.4	35.4	34.9	34.2	33.1	32.0	30.6	29.0		
Decreasing	15 USG 9-15	0.800	111	2.66	69.4	21.0	31.8	37.0	2 Samples	0.040	0.069	0.156	0.302	0.590	1.17	2.33	4.36		
										42.5	42.5	42.3	41.4	40.4	38.3	35.2	31.8		
	15 USG 9-15	0.805	113	2.66	77.1	30.5	30.5	34.0		0.044	0.073	0.160	0.305	0.595	1.18	2.34	4.36		
										37.1	37.0	36.8	36.4	35.7	34.6	32.8	30.5		
↓																			

Consolidation Test Data Summary

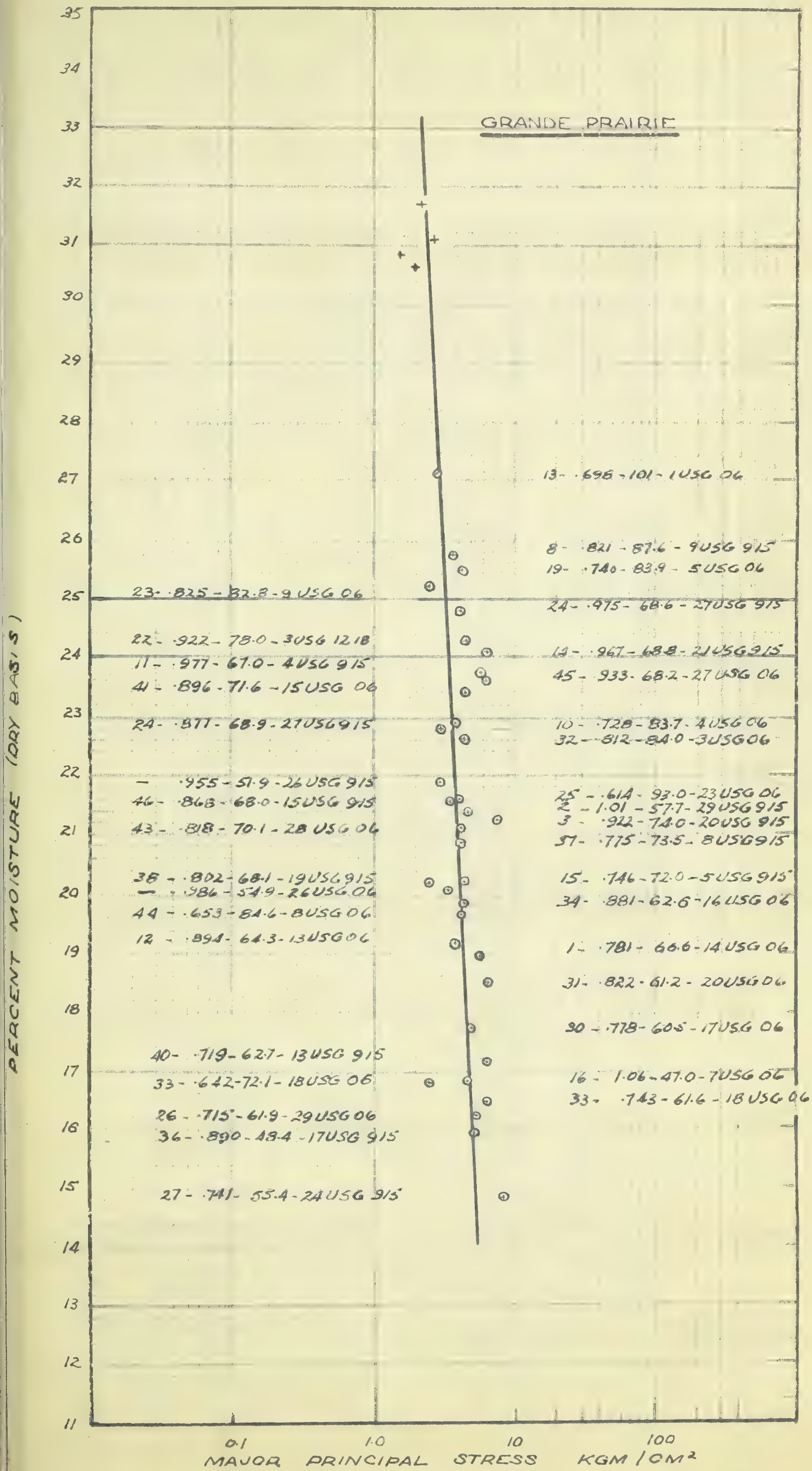
Soils Tested Grand Prairie

Increasing ← Plasticity → Decreasing

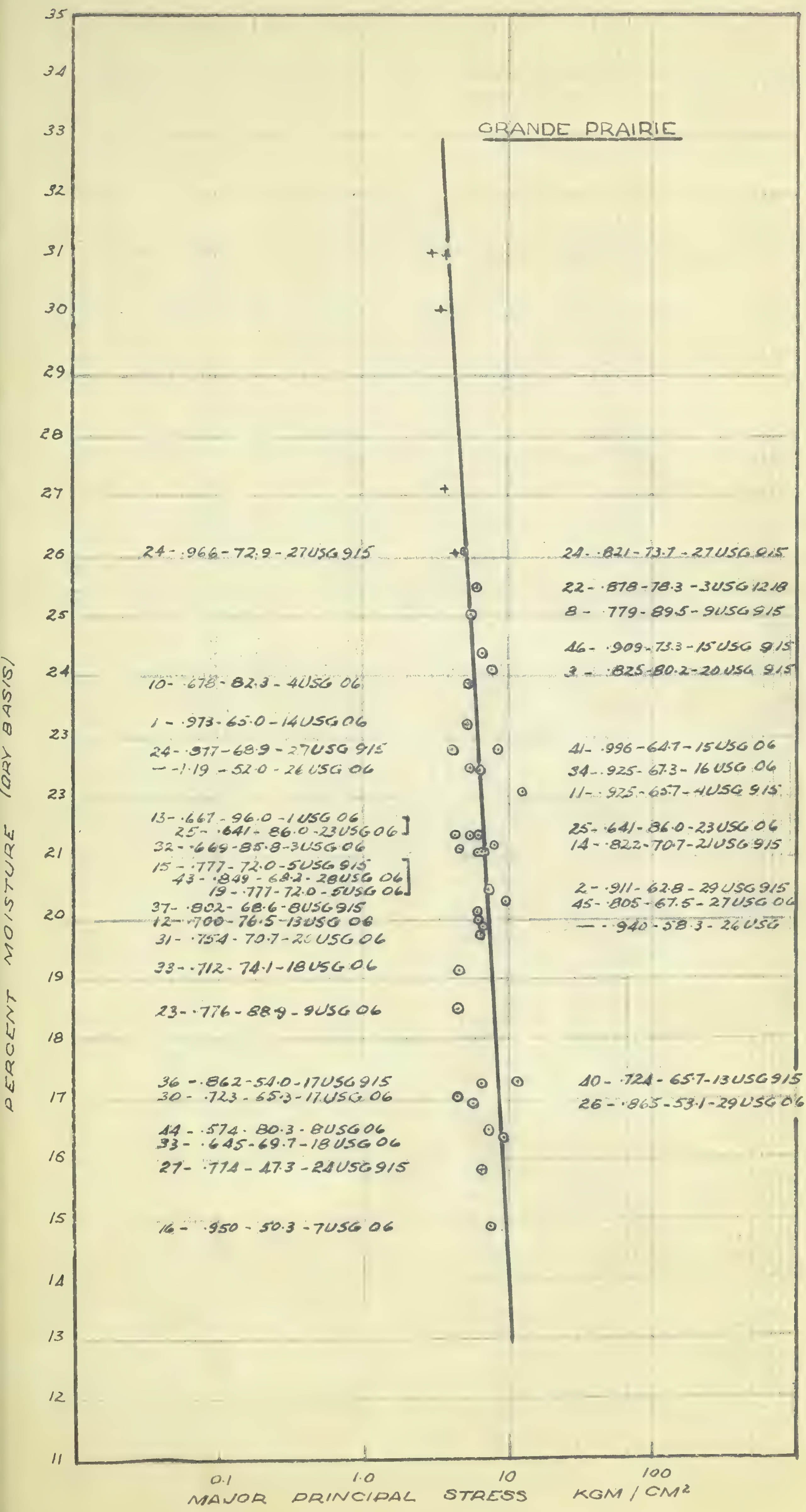
Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
28 USG 0-6	0.633	124	2.69	90.2	21.4	21.8	23.4		0.044	0.073	0.159	0.305	0.595	1.18	2.34			
									24.4	24.4	24.5	24.3	23.6	23.0	21.8			
26 USG 0-6	0.897	104	2.67	57.7	19.6	29.0	36.3	No Plasticity Number	0.035	0.070	0.141	0.295	0.590	1.18	2.36	4.73		
									38.7	38.6	38.1	37.5	36.3	34.3	32.2	29.0		
27 USG 9-15	1.13	99	2.69	61.3	42.0	36.8	43.6	"	0.035	0.095	0.141	0.281	0.562	1.12	2.25			
									44.7	44.3	43.9	42.1	39.0	35.7	32.4			
27 USG 9-15	--	--	2.69	--	30.3	28.0	31.4	" 2 Samples	0.048	0.073	0.158	0.306	0.600	1.19	2.37	4.43		
									34.9	34.9	34.8	34.9	33.9	32.9	30.6	28.0		



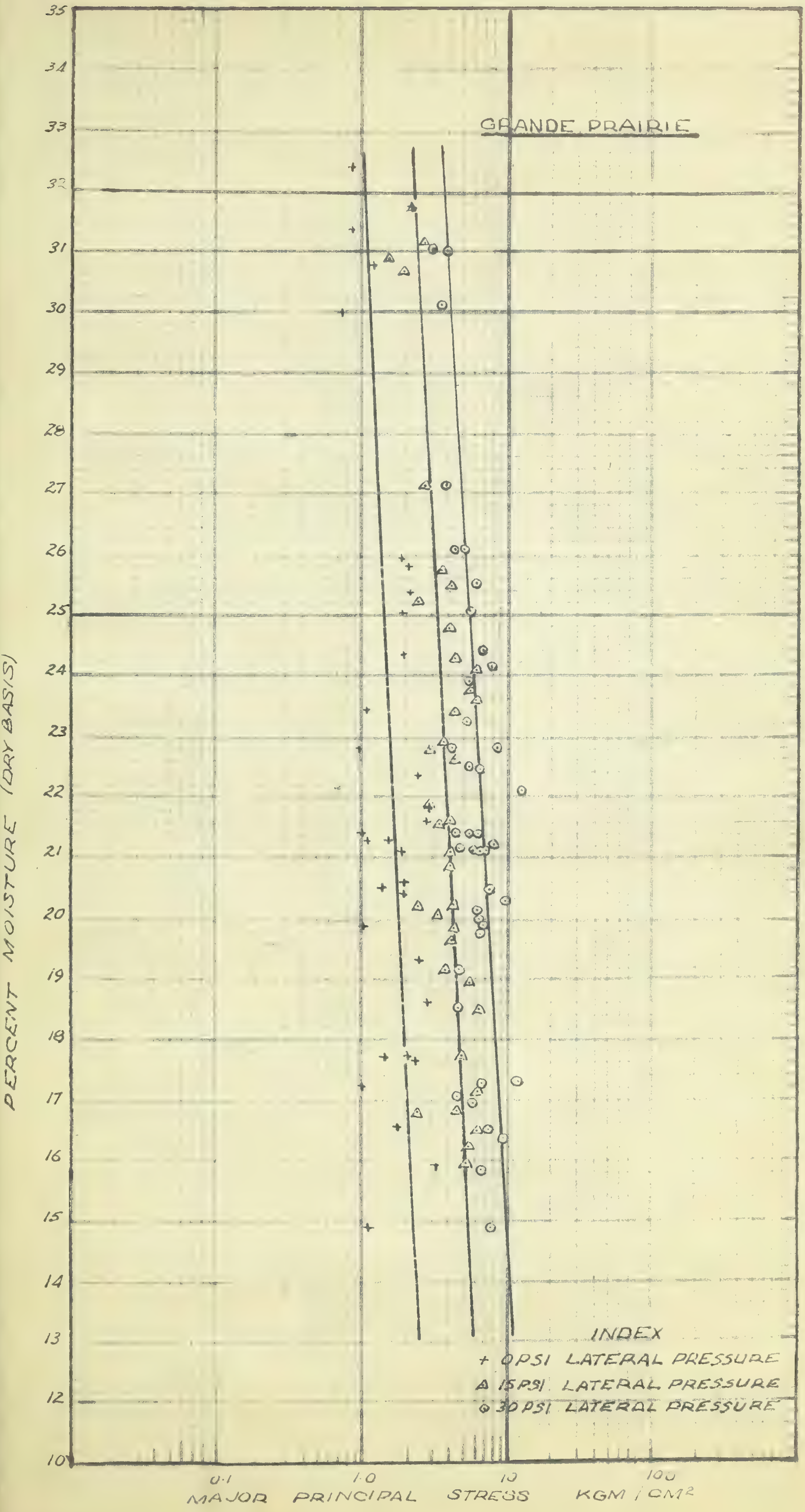
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 0 - LATERAL PRESSURE



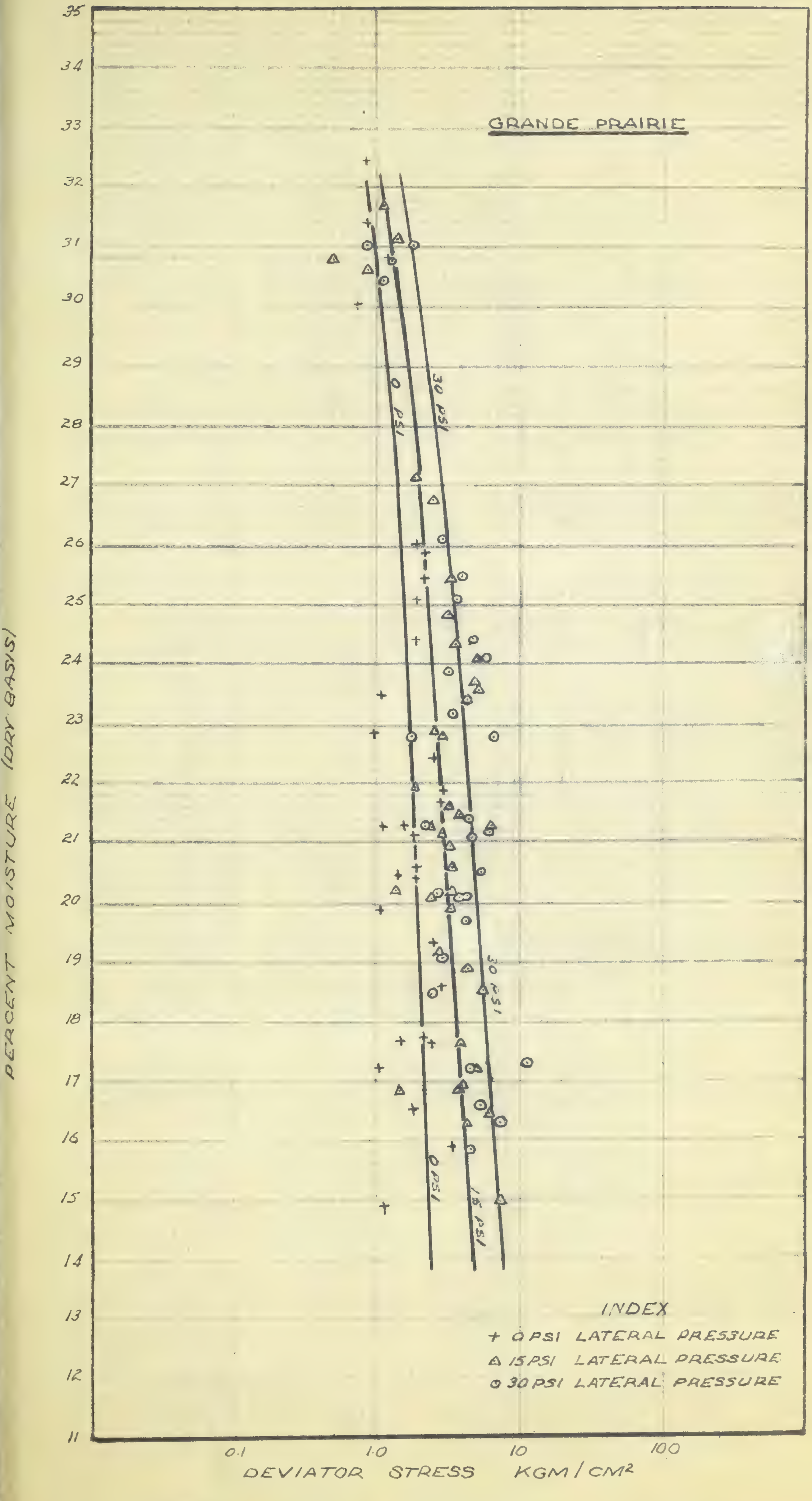
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 15 PSI LATERAL PRESSURE



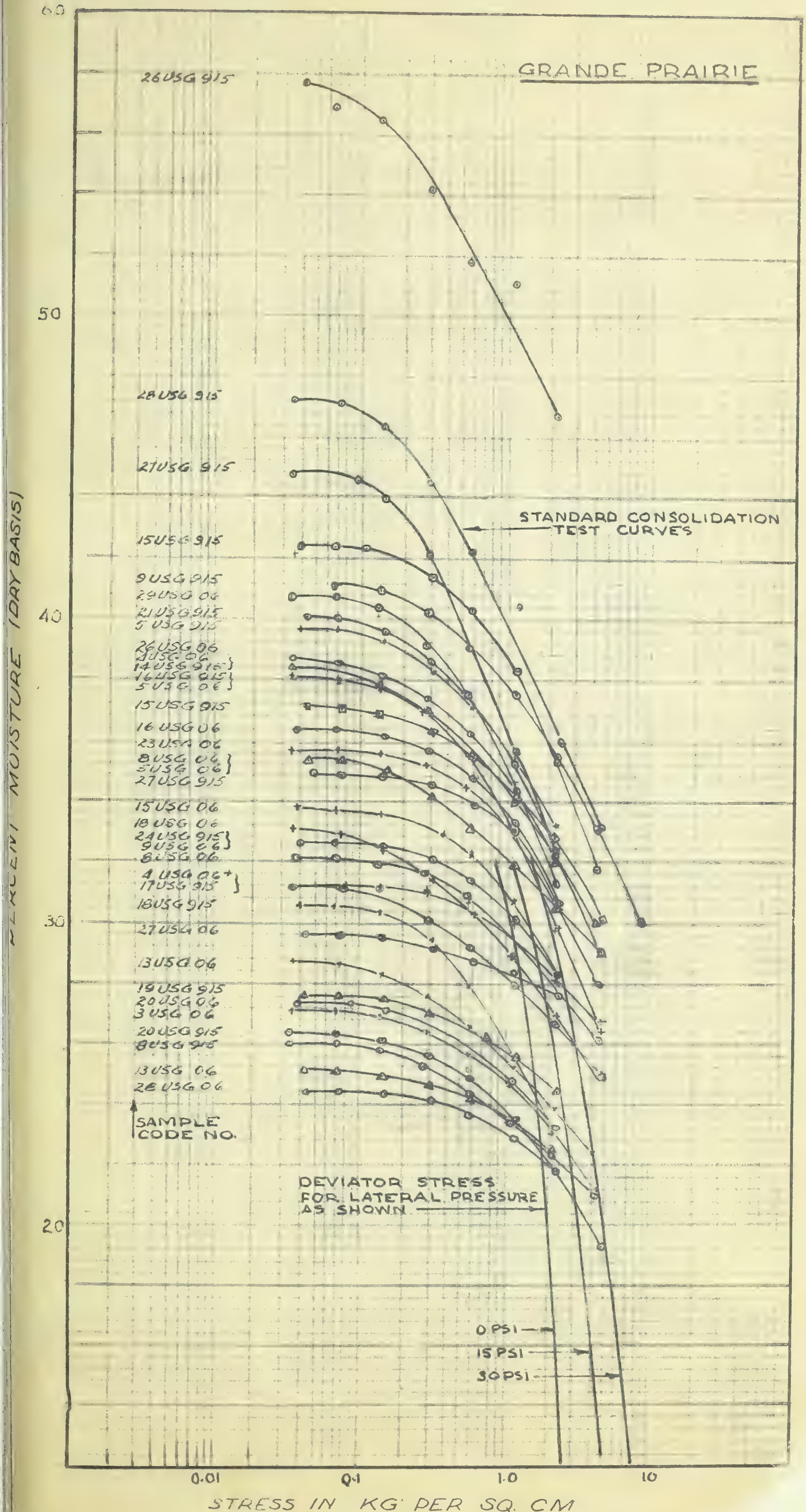
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 30 PSI LATERAL PRESSURE



PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS

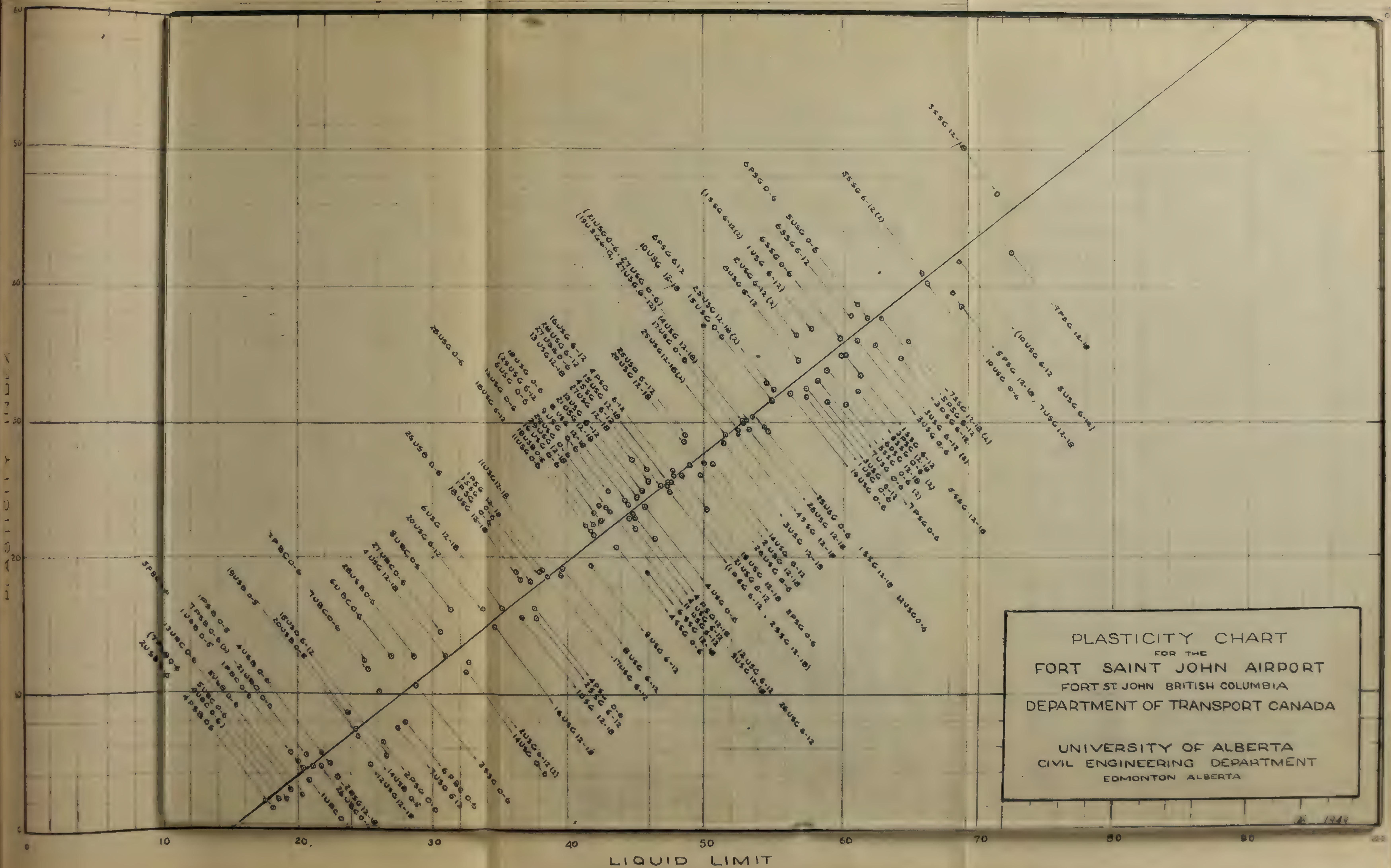


PERCENT MOISTURE (DRY BASIS) AT END OF TEST VS DEVIATOR STRESS



CONSOLIDATION AND TRIAXIAL TEST
MOISTURE - STRESS RELATIONS

FORT ST. JOHN


PLASTICITY CHART
FOR THE
FORT SAINT JOHN AIRPORT
FORT ST. JOHN BRITISH COLUMBIA
DEPARTMENT OF TRANSPORT CANADA
UNIVERSITY OF ALBERTA
CIVIL ENGINEERING DEPARTMENT
EDMONTON ALBERTA
LIQUID LIMIT
PLASTICITY INDEX
0
10
20
30
40
50
60
70
80
90

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED: Degree Saturation %	UNCONFINED: % Moisture Start	UNCONFINED: % Moisture End	UNCONFINED: Void Ratio Start	UNCONFINED: Deviator Str. Kgm/cm²	15 p.s.i. LATERAL PRESSURE: Degree Saturation %	15 p.s.i.: % Moisture Start	15 p.s.i.: % Moisture End	15 p.s.i.: Void Ratio Start	15 p.s.i.: Max. Major Principal Stress Kgm/cm²	15 p.s.i.: Deviator Stress Kgm/cm²	30 p.s.i. LATERAL PRESSURE: Degree Saturation %	30 p.s.i.: Percent Moisture Start	30 p.s.i.: Percent Moisture End	30 p.s.i.: Void Ratio Start	30 p.s.i.: Max. Major Principal Stress Kgm/cm²	30 p.s.i.: Deviator Stress Kgm/cm²	'C' Cohesion Kgm/cm²	Angle of Internal Friction
1	7 USG 12-18	68.8	38.6	2.70	89.5	24.2	--	--	1.36	87.8	25.1	--	--	2.97	1.91	92.9	25.3	--	--	3.98	1.87	0.52	13
2	6 SSG 0-6	59.8	36.2	2.71	97.7	22.5	21.7	0.625	1.69	38.8	23.2	23.1	0.705	2.93	1.87	98.4	21.1	20.9	0.582	4.60	2.49	0.77	8.5
4	7 PSG 0-6	59.0	31.4	2.68	97.7	27.0	27.7	0.723	1.10	98.0	26.8	26.4	0.715	2.47	1.41	100.8	26.4	26.1	0.681	4.21	2.10	0.47	11.5
5	19 USG 9-15	56.2	32.0	2.71	94.1	23.2	23.0	0.700	1.95	93.1	23.1	43.0	0.673	3.60	2.54	94.1	21.7	21.3	0.626	4.91	2.80	1.14	6
6	15 USG 9-15	54.9	31.5	2.66	85.1	19.7	19.4	0.627	2.45	78.8	19.7	19.4	0.676	4.51	3.45	78.6	19.8	19.5	0.685	6.07	3.96	1.22	4.5
7	25 USG 0-6	53.8	30.4	2.69	87.7	20.1	19.9	0.618	1.94	80.0	19.3	19.2	0.650	4.24	3.18	96.5	21.0	20.9	0.587	4.63	2.52	0.67	22.5
8	12 USG 9-15	54.7	29.4	2.70	92.9	20.5	20.2	0.579	2.44	94.1	19.4	19.1	0.553	4.59	3.43	92.6	20.0	19.7	0.580	--	--	--	--
9	19 USG 9-15	43.0	29.9	2.68	85.8	16.4	16.1	0.514	4.40	75.6	16.2	16.2	0.575	6.70	5.64	85.8	18.1	18.0	0.528	6.83	4.72	1.52	19.5
10	27 USG 10-16	52.8	30.0	2.66	75.8	19.9	19.5	0.715	1.93	78.7	20.4	20.2	0.702	4.44	3.38	81.3	19.5	19.5	0.714	6.36	4.25	0.7	20.5
11	21 USG 0-6	53.0	30.3	2.72	75.1	18.6	18.1	0.676	3.39	77.4	20.8	20.4	0.730	4.61	3.55	76.3	19.2	19.1	0.684	7.22	5.11	1.27	17
12	14 USG 10-16	52.8	30.1	2.68	90.6	22.1	--	0.663	1.25	85.0	22.5	--	0.644	2.28	1.22	91.1	22.1	--	0.654	3.33	1.22	0.50	11
13	17 USG 0-6	53.2	30.1	2.69	91.3	21.5	21.2	0.633	1.80	83.7	20.6	--	0.661	3.07	2.01	84.5	22.3	22.2	0.710	4.06	1.95	0.83	5
14	26 USG 10-16	53.2	29.4	2.72	93.0	22.3	--	0.628	1.63	96.0	20.8	--	0.555	3.11	2.05	86.0	20.4	--	0.644	4.08	1.97	0.70	11
15	17 USG 9-15	52.7	29.4	2.70	91.2	18.4	18.1	0.545	3.36	85.9	17.3	17.1	0.545	5.66	4.60	103.5	17.0	16.7	--	6.44	4.33	1.41	11
16	26 USG 0-6	36.4	19.0	2.69	100	22.7	--	0.611	1.35	100	23.5	--	0.628	2.41	1.35	99.0	22.1	--	0.598	3.64	1.53	0.65	30
17	16 USG 9-15	47.8	26.2	2.71	74.0	15.8	15.6	0.576	6.83	82.2	15.3	14.8	0.499	10.26	9.20	63.0	15.3	15.0	0.653	11.03	8.92	--	--
18	26 USG 10-16	46.4	24.9	2.67	93.0	22.3	--	0.632	1.63	96.0	20.8	--	0.557	3.11	2.05	86.0	20.4	--	0.643	4.08	1.97	0.70	11

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED--Fort St. John

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED: Degree Saturation %	UNCONFINED: % Moisture Start	UNCONFINED: % Moisture End	UNCONFINED: Void Ratio Start	UNCONFINED: Deviator Str. Kgm/cm^2	15 p.s.i. LATERAL PRESSURE: Degree Saturation %	15 p.s.i.: % Moisture Start	15 p.s.i.: % Moisture End	15 p.s.i.: Void Ratio Start	15 p.s.i.: Max. Major Principal Stress Kgm/cm^2	15 p.s.i.: Deviator Stress Kgm/cm^2	30 p.s.i. LATERAL PRESSURE: Degree Saturation %	30 p.s.i.: Percent Moisture Start	30 p.s.i.: Percent Moisture End	30 p.s.i.: Void Ratio Start	30 p.s.i.: Max. Major Principal Stress Kgm/cm^2	30 p.s.i.: Deviator Stress Kgm/cm^2	'C' Cohesion Kgm/cm^2	Angle of Internal Friction
19	13 USG 9-15	45.3	24.9	2.60	52.9	14.7	14.2	0.800	5.37	60.2	15.1	14.9	0.802	10.21	9.15	47.4	14.5	14.2	--	10.55	8.44	--	--
20	18 USG 0-6	44.3	24.2	2.69	82.6	17.2	16.9	0.558	2.95	80.9	18.0	17.8	0.590	4.83	3.77	83.3	16.3	16.1	0.528	7.29	5.18	1.04	21
21	9 USG 0-6	49.1	22.6	2.69	93.7	20.8	--	0.598	1.61	94.5	21.7	--	0.620	2.41	3.47	93.5	19.3	--	0.566	4.79	2.68	0.60	16
22	16 USG 0-6	42.3	22.7	2.69	60.4	14.7	14.3	0.652	2.94	76.0	14.5	14.2	0.514	8.64	7.58	73.4	14.9	14.3	0.545	7.99	5.88	--	--
23	20 USG 10-16	42.0	22.4	2.70	86	20.2	--	0.632	1.78	95	21.1	--	0.596	4.32	3.26	94.5	19.1	--	0.541	5.20	3.09	0.59	24
24	11 USG 10-16	39.7	18.7	2.70	74.8	18.4	18.2	0.666	1.95	81.6	19.5	19.3	0.649	4.34	3.28	78.4	18.7	18.6	0.648	3.61	1.50	0.75	17
25	18 USG 10-16	36.6	18.5	2.68	75	18.7	18.2	0.668	3.48	83.2	18.9	18.6	0.613	3.71	2.65	85.5	18.3	18.2	0.578	6.24	4.13	1.50	8
26	14 USG 0-6	32.3	11.5	2.68	81.6	19.6	19.3	0.652	7.48	--	--	--	--	--	--	82.6	21.7	20.7	0.713	5.19	3.08	0.58	15.5

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED-- Fort St. John

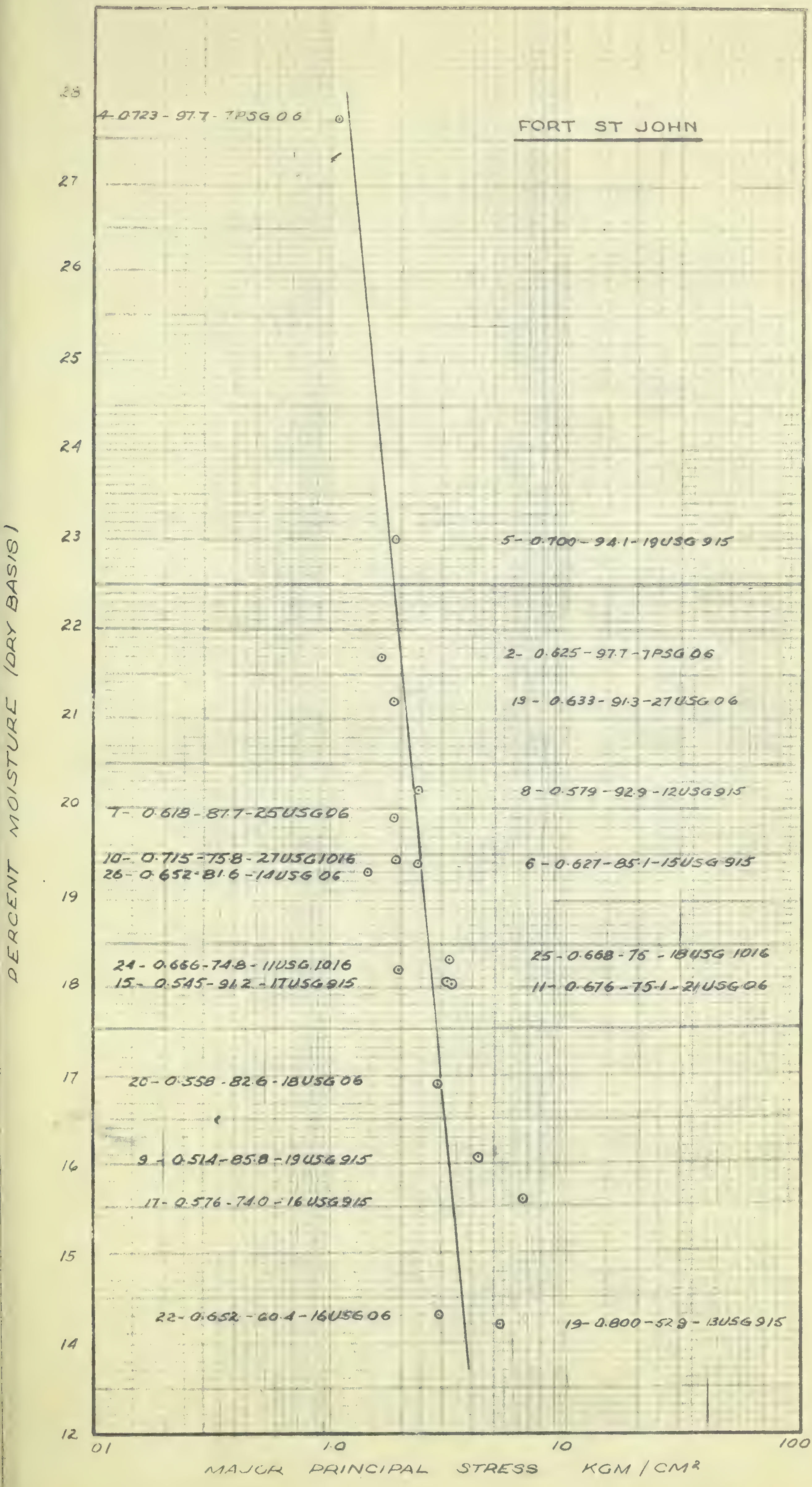
	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)								
↑	10 USG 10-16	0.595	127	2.69	96.5	21.4	22.2	24.9		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035	
										25.12	24.96	24.25	23.85	23.07	21.99	20.27	22.2	
Increasing	10 USG 0-6	0.809	119	2.72	94.5	28.1	26.9	32.6		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035	
										35.99	35.99	35.73	35.18	34.11	32.40	30.04	26.93	
	5 USG 0-6	0.698	124	2.70	89.0	22.8	29.5	25.1		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035	
										29.59	29.59	29.41	28.92	28.14	26.68	24.72	29.50	
	6 SSG 0-6	0.644	126	2.71	92.1	21.8	24.1	25.8		0.040	0.070	0.156	0.301	0.591	1.17	2.33		
										28.19	28.19	28.04	27.43	26.62	25.44	24.1		
Plasticity	15 USG 0-6	0.539	128	2.68	88.8	17.8	19.4	21.5		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035	
										21.62	21.57	21.42	21.11	20.64	19.98	18.97	19.40	
	25 USG 0-6	0.523	130	2.69	91.9	17.9	19.8	22.6		0.044	0.073	0.160	0.305	0.590	1.17	2.34	4.36	8.70
										24.56	24.56	24.48	24.35	23.81	23.28	22.44	21.45	19.76
	21 USG 0-6	0.682	122	2.72	83.7	21.0	23.9	25.1		0.035	0.070	0.141	0.281	0.562	1.12	2.25		
										26.30	26.27	26.21	26.02	25.43	24.77	23.9		
	27 USG 0-6	0.562	127	2.69	87.0	18.2	19.9	21.5		0.044	0.074	0.163	0.310	0.605	1.19	2.37	4.43	
										23.34	23.34	23.28	23.04	22.60	21.94	21.07	19.9	
Decreasing	(12-18) 26 USG 10-16	0.569	127	2.69	98.0	20.9	23.7	23.9		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035	
										23.91	23.92	23.89	23.79	23.59	23.27	22.81	23.7	
	(12-18) 14 USG 10-16	0.682	123	2.69	87.1	22.2	24.8	25.9		0.040	0.065	0.131	0.262	0.525	1.05	2.10	4.20	0.040
										26.67	26.61	26.44	26.08	25.10	24.49	22.79	20.70	24.8
↓	17 USG 0-6	0.588	128	2.70	86.3	18.5	21.1	21.9		0.040	0.070	0.156	0.301	0.591	1.17	2.33		
										23.45	23.43	23.27	22.94	22.47	21.90	21.1		

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing →	20 USG 0-6	0.623	123	2.64	90.0	21.3	25.9	28.7		0.040	0.070	0.156	0.301	0.591	1.17	2.33	4.36	8.70	
										34.02	34.00	33.70	33.14	32.15	31.00	29.60	28.16	25.94	
	26 USG 0-6	0.749	122	2.69	100.0	27.8	29.5	30.8		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035		
										30.44	30.44	30.29	29.92	29.05	27.71	26.05	29.5		
	(12-18) 19 USG 9-15	0.644	124	2.68	93.4	22.6	25.0	27.0		0.039	0.068	0.154	0.296	0.583	1.15	2.29			
										29.04	29.00	28.91	28.53	27.76	26.53	25.0			
	(6-12) 16 USG 9-15	0.510	129	2.69	86.0	16.3	19.7	22.0		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.30		
										26.16	26.16	25.92	25.40	24.28	22.87	21.28	19.7		
Plasticity	(12-18) 15 USG 9-15	0.609	114.5	2.68	65.0	15.5	19.9	24.9		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.50		
										25.97	25.95	25.89	25.58	25.17	24.46	22.58	19.9		
	(12-18) 13 USG 9-15	0.495	128	2.65	86.1	16.1	20.2	25.5		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.25	8.56	
										26.67	26.65	26.45	26.00	25.23	24.21	22.93	21.70	20.2	
	18 USG 0-6	0.476	133	2.69	96.5	17.1	18.1	19.3		0.040	0.070	0.156	0.301	0.591	1.17	2.33	4.36		
										20.69	20.68	20.64	20.48	20.21	19.74	19.01	18.1		
	(12-18) 12 USG 9-15	0.524	123	2.66	64.5	12.8	18.5	20.3		0.040	0.070	0.156	0.301	0.591	1.17	2.33			
										20.64	20.62	20.41	20.20	19.79	19.15	18.52			
Decreasing ←	9 USG 0-6	0.605	125	2.69	89.5	20.0	22.1	22.4		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035		
										22.61	22.80	22.76	22.48	21.99	21.31	20.25	22.1		
	4 USG 9-15	0.456	133	2.69	88.5	14.9	18.1	18.7		0.040	0.070	0.156	0.301	0.591	1.17	2.33			
										19.82	19.81	19.75	19.45	19.15	18.65	18.08			
	29 USG 0-6	0.508	133	2.74	91.8	16.9	18.1	19.8		0.044	0.073	0.160	0.305	0.590	1.17	2.34	4.39		
										21.85	21.84	21.66	21.36	20.82	20.10	19.16	18.1		

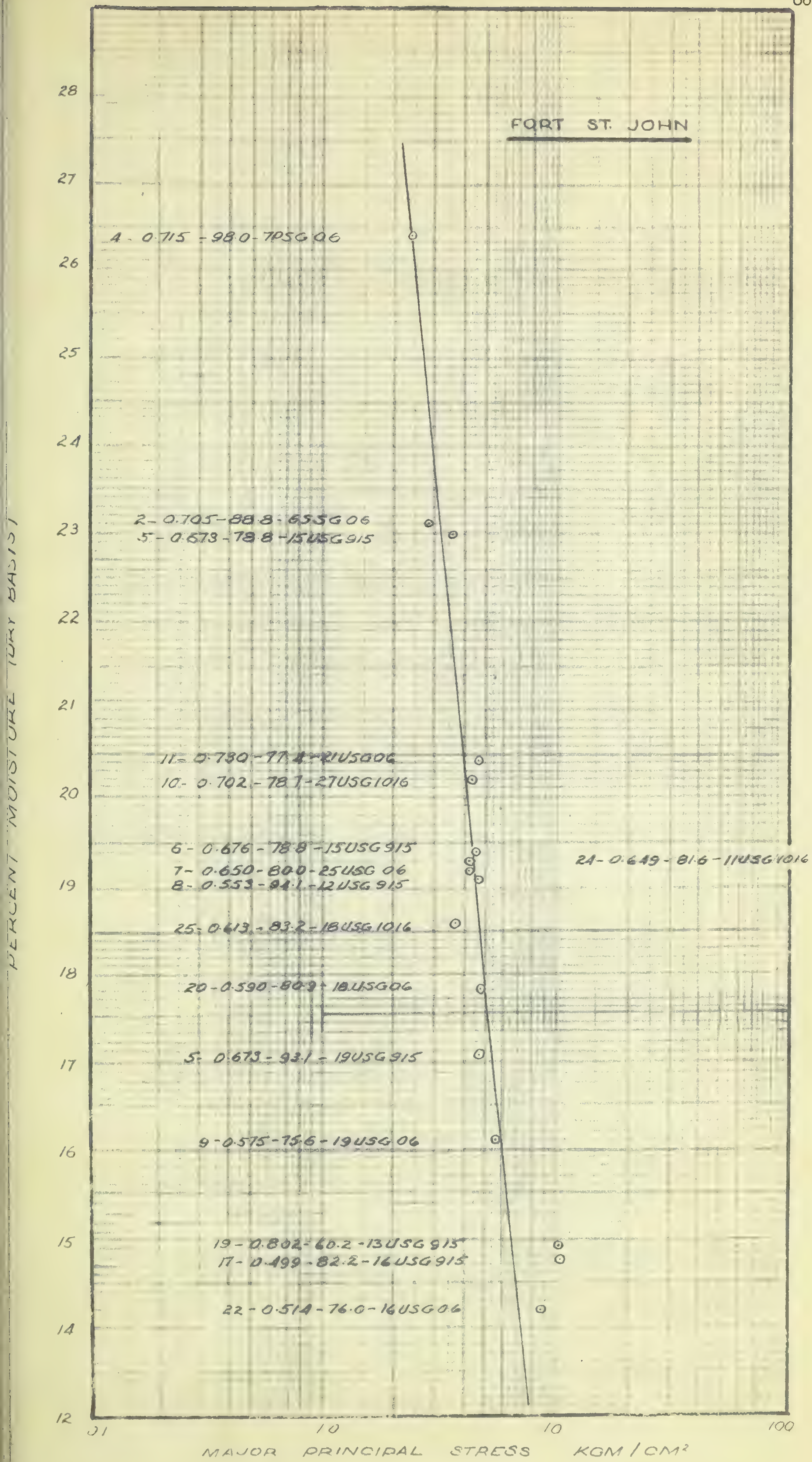
Consolidation Test Data Summary

Soils Tested ..Fort St. John.......

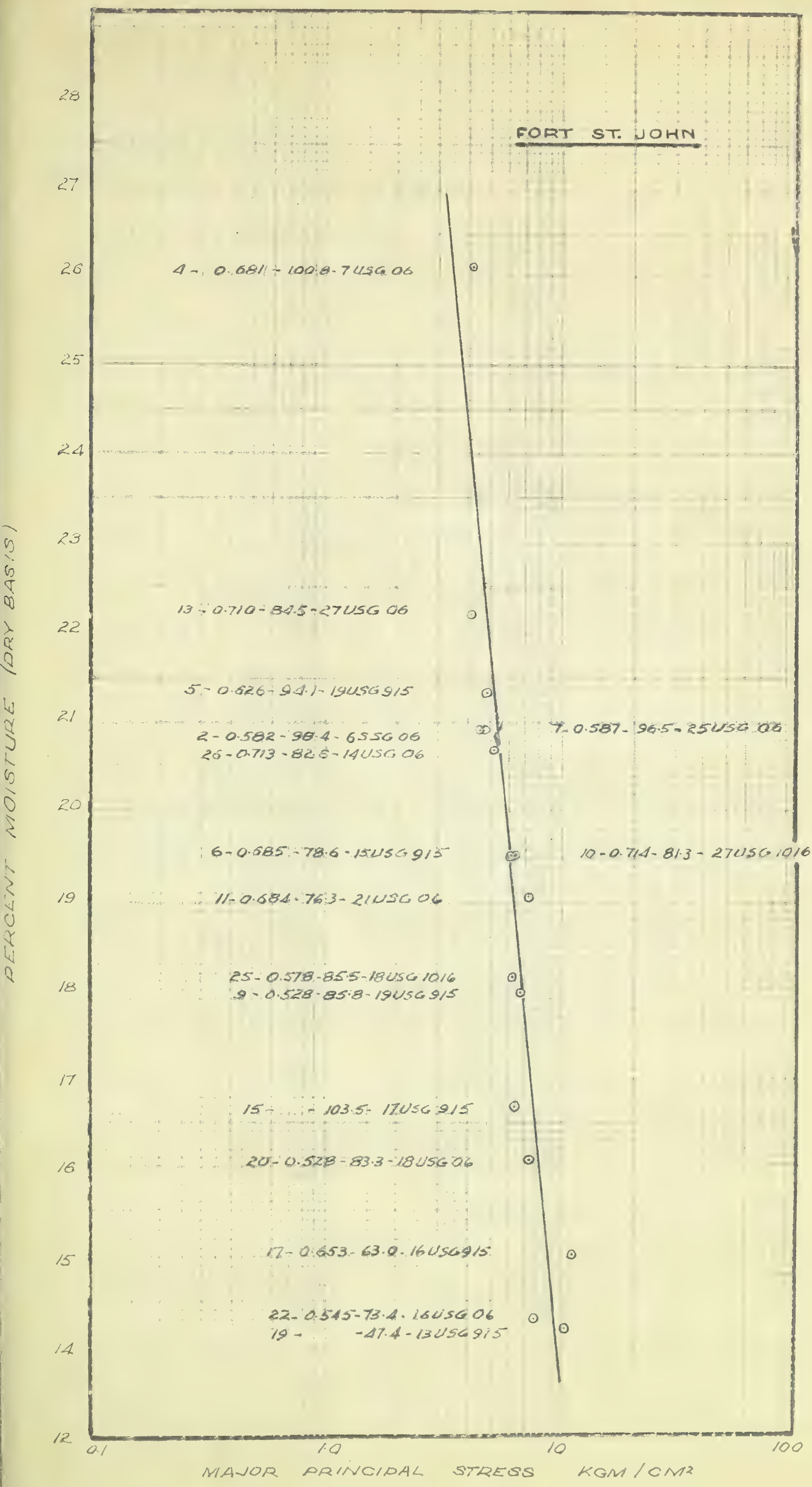
Plasticity	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)							
Increasing ↑	(12-18) 29 USG 10-16	0.489	129	2.67	85.0	15.6	22.8	23.8		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.50
										27.30	27.20	26.88	26.30	25.54	24.65	23.75	22.84
	13 USG 0-6	0.537	126	2.69	76.5	15.3	20.6	21.5		0.035	0.070	0.141	0.281	0.562	1.12	2.25	0.035
										20.77	21.07	21.02	20.89	20.48	19.85	18.73	20.6
	16 USG 0-6	0.575	125	2.69	76.3	16.1	21.2	23.6		0.035	0.070	0.141	0.281	0.562	1.12	2.25	4.30
										28.55	28.53	28.22	27.46	26.51	24.75	23.06	21.2
	11 USG 0-6	0.689	118	2.72	66.9	17.0	21.8	25.2		0.040	0.070	0.156	0.301	0.591	1.17	2.33	
										25.58	25.54	25.46	25.14	24.39	23.24	21.82	
Plasticity	(6-12) 9 USG 9-15	0.575	124	2.67	80.1	17.3	23.1	24.1		0.044	0.072	0.157	0.300	0.585	1.12	2.25	
										26.72	26.69	26.47	25.96	25.10	24.23	23.05	
	(6-12) 17 USG 9-15	0.510	131	2.70	90.4	17.0	17.8	19.3		0.035	0.072	0.157	0.300	0.585	1.12	2.25	4.50
										20.70	20.72	20.60	20.35	19.92	19.29	18.61	17.8
	14 USG 0-6	0.701	124	2.68	98.0	25.6	25.3	27.3		0.035	0.070	0.141	0.281	0.562	1.12	2.25	
										28.20	28.18	28.08	27.84	27.43	26.61	25.30	
Decreasing ↓																	



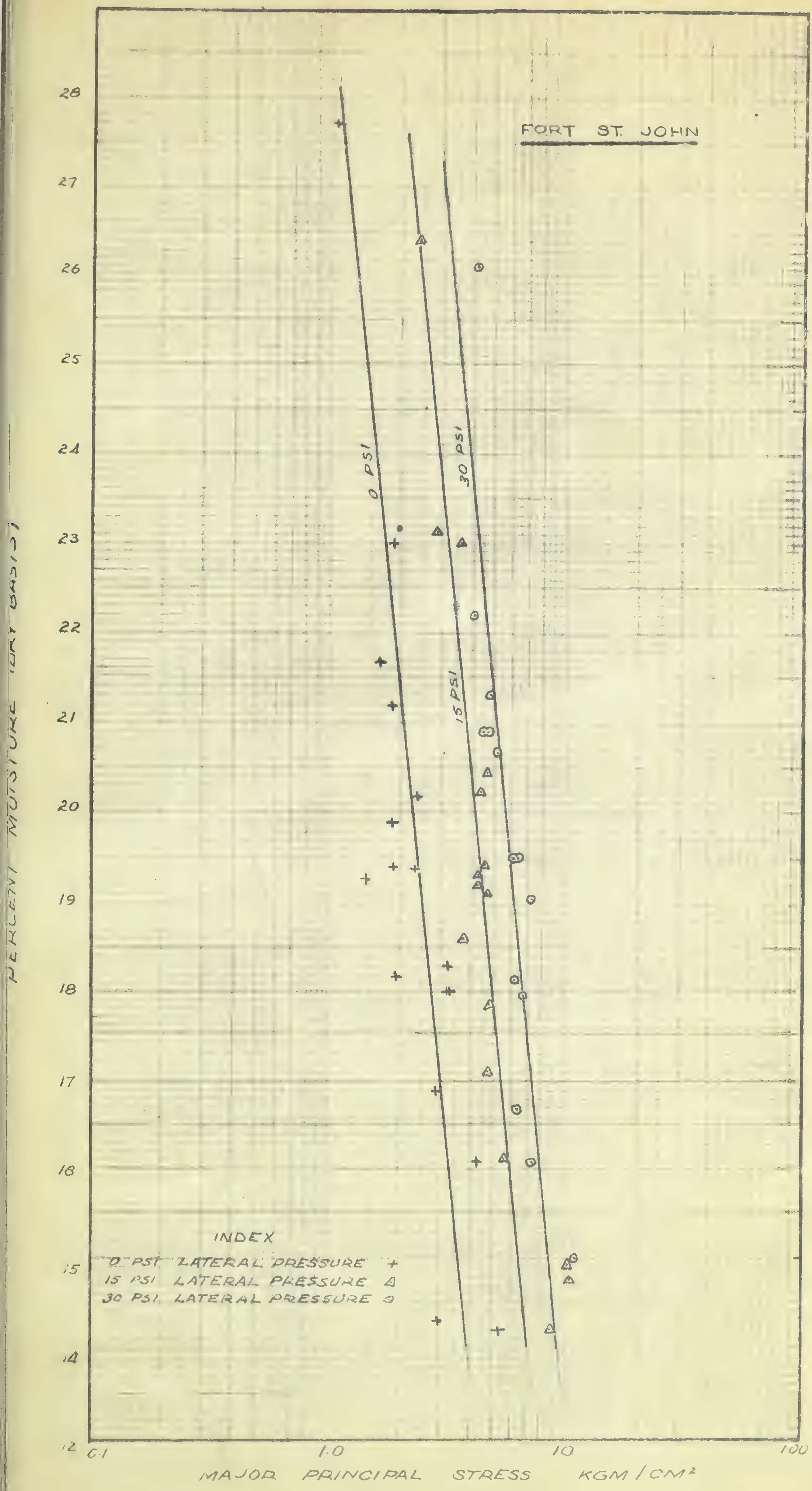
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 0 LATERAL PRESSURE



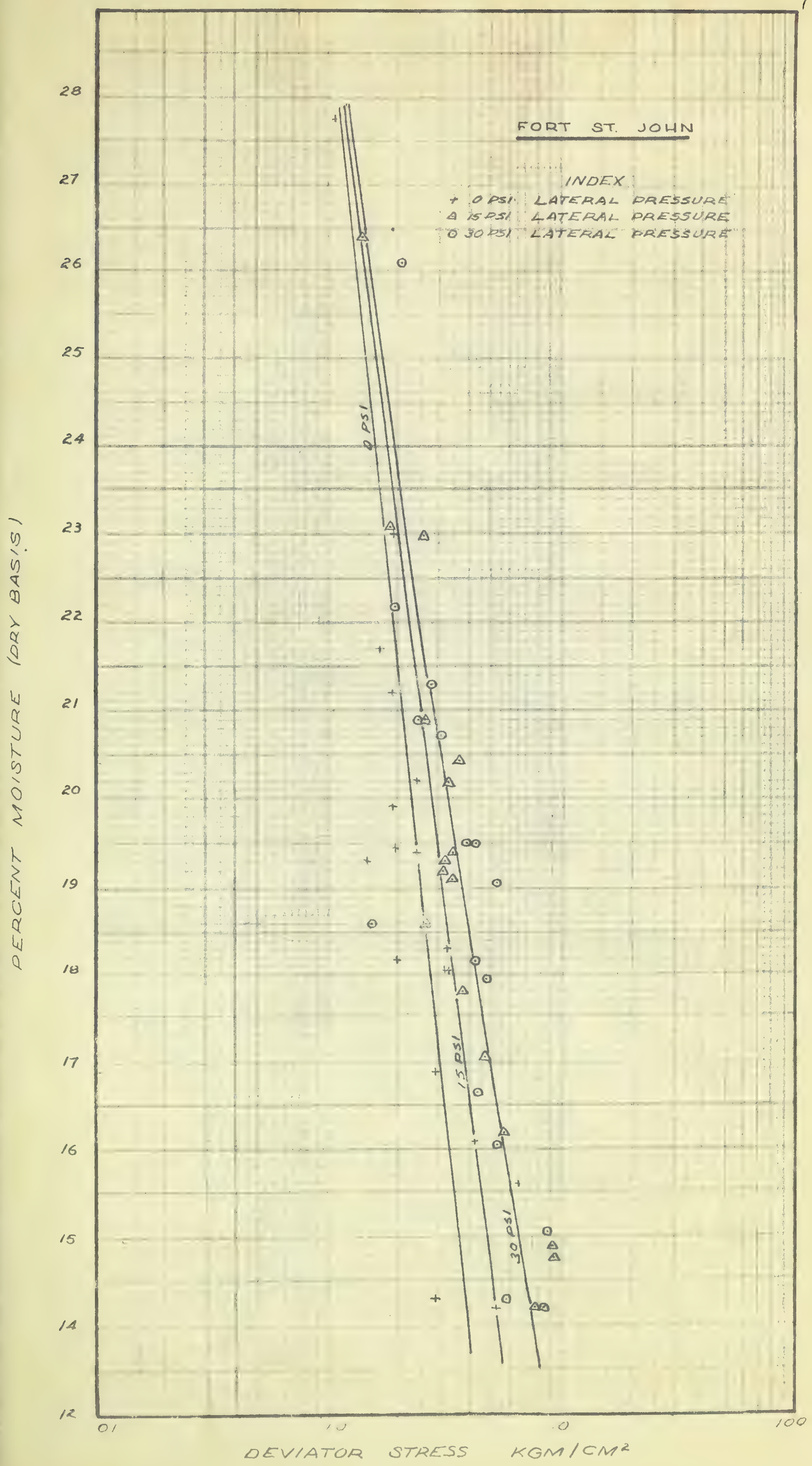
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 15 PSI LATERAL PRESSURE



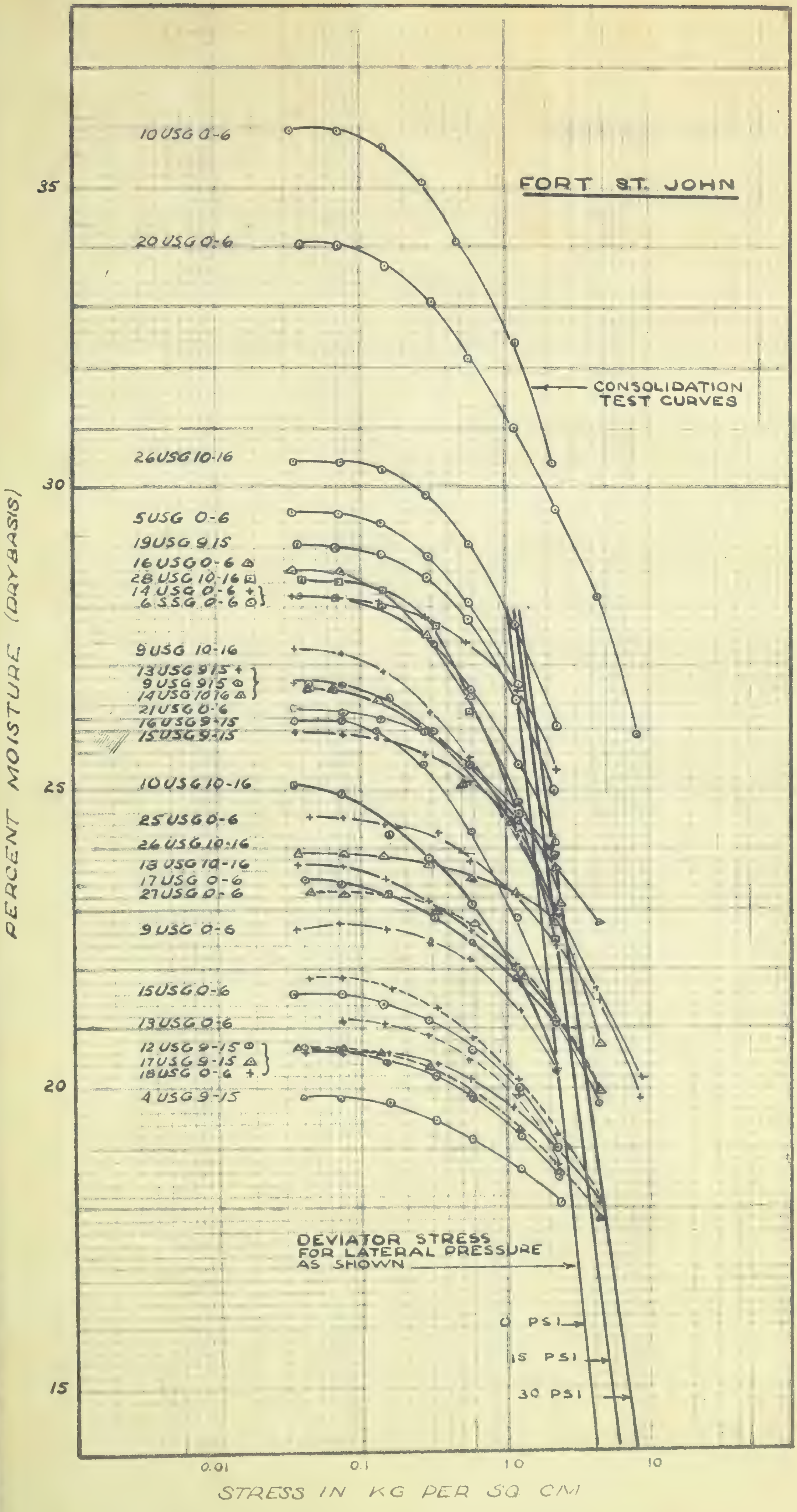
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS. MAJOR PRINCIPAL STRESS - 30 PSI LATERAL PRESSURE



PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS

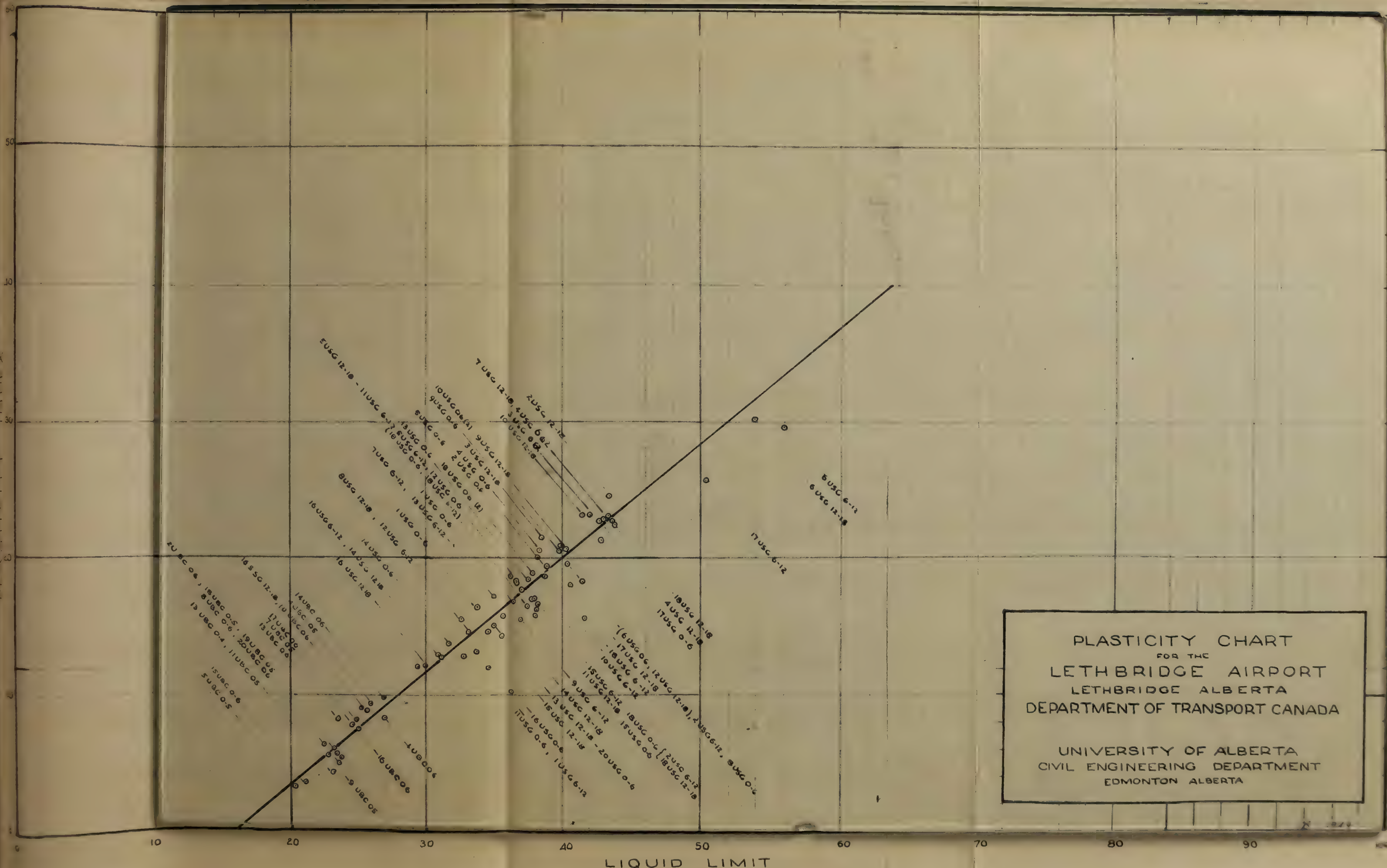


PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS. DEVIATOR STRESS



CONSOLIDATION AND TRIAXIAL TEST
MOISTURE - STRESS RELATIONS

LETHBRIDGE


PLASTICITY CHART
FOR THE
LETHBRIDGE AIRPORT
LETHBRIDGE ALBERTA
DEPARTMENT OF TRANSPORT CANADA
UNIVERSITY OF ALBERTA
CIVIL ENGINEERING DEPARTMENT
EDMONTON ALBERTA
0
10
20
30
40
50
60
70
80
90
LIQUID LIMIT

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED -- Lethbridge

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED: Degree Saturation %	UNCONFINED: % Moisture Start	UNCONFINED: % Moisture End	UNCONFINED: Void Ratio Start	UNCONFINED: Deviator Str. Kgm/cm^2	15 p.s.i. LATERAL PRESSURE: Degree Saturation %	15 p.s.i.: % Moisture Start	15 p.s.i.: % Moisture End	15 p.s.i.: Void Ratio Start	15 p.s.i.: Max. Major Principal Stress Kgm/cm^2	15 p.s.i.: Deviator Stress Kgm/cm^2	30 p.s.i. LATERAL PRESSURE: Degree Saturation %	30 p.s.i.: Percent Moisture Start	30 p.s.i.: Percent Moisture End	30 p.s.i.: Void Ratio Start	30 p.s.i.: Max. Major Principal Stress Kgm/cm^2	30 p.s.i.: Deviator Stress Kgm/cm^2	'C' Cohesion Kgm/cm^2	Angle of Internal Friction
1	6 USG 9-15	56.0	29.7	2.70	56.0	19.7	19.2	--	1.08	46	17.9	17.7	--	6.06	5.00	58	16.9	16.9	--	12.5	10.4	0.3	43
2	17 USG 9-15	50.4	25.7	2.69	42	13.5	13.3	--	0.50	37	12.6	12.0	--	5.70	4.64	40	13.6	13.5	--	9.97	7.86	0.5	35
3	3 USG 0-6	43.0	23.0	2.69	48	14.3	13.7	0.668	1.47	44	14.2	13.8	0.840	6.64	5.58	46	14.3	13.5	0.801	10.4	8.26	0.25	35
4	10 USG 12-18	41.6	23.1	2.67	36	13.6	13.0	--	1.90	36	13.6	13.2	--	5.20	4.14	39	12.8	13.0	--	10.9	8.83	0.3	43
5	17 USG 0-6	43.1	21.3	2.65	69	14.0	14.2	0.533	1.97	47	15.3	17.2	0.895	4.34	3.28	48	15.4	19.7	0.799	5.65	3.54	0.5	35
6	9 USG 9-15	40.0	20.9	2.53	37	12.1	11.3	0.885	1.68	39.4	15.0	14.3	1.00	5.57	4.51	35.5	12.3	12.9	0.924	9.71	7.60	0.4	36
7	2 USG 0-6	38.9	19.5	2.69	32.7	10.9	10.4	--	3.02	34.7	11.2	11.0	--	8.70	7.64	44.6	12.2	11.8	--	12.32	10.22	0.7	42
8	8 USG 0-6	41.5	18.4	2.70	57	17.5	16.2	0.815	2.78	33	17.3	16.2	1.40	4.78	3.72	42	17.7	20.6	1.10	5.32	3.21	1.0	18
9	10 USG 0-6	39.9	20.3	2.69	43	13.8	13.7	0.980	0.64	50	14.5	16.4	0.938	5.36	4.30	45	13.9	18.0	0.827	4.98	2.87	0.9	10
10	1 USG 0-6	36.3	16.8	2.68	51.6	12.3	11.9	0.648	5.90	50	11.9	11.5	0.641	7.45	6.39	53.7	12.2	--	0.614	9.92	7.81	2.2	17.5
11	15 USG 9-15	34.6	14.7	2.68	52.8	16.0	15.5	--	1.25	59	16.5	16.2	--	6.11	5.05	61.5	16.0	15.9	--	9.42	7.30	0.9	37
12	13 USG 0-6	37.5	18.5	2.64	49	13.0	12.7	0.706	2.13	51	13.3	13.8	0.676	5.73	4.67	59	13.2	--	0.594	5.55	3.44	0.75	18
12	13 USG 0-6	37.5	18.5	2.64	42.1	12.9	12.3	0.821	1.60	38.4	12.8	12.4	0.896	6.70	5.64	40.7	12.8	12.7	0.848	10.00	7.89	0.4	41.5
13	12 USG 0-6	37.1	17.9	2.60	23.5	9.26	8.73	--	1.67	25.1	8.97	8.72	--	8.41	7.35	30.0	11.0	10.0	--	11.85	9.74	0.4	45
14	14 USG 9-15	35.5	17.8	2.68	51	17.8	--	--	0.51	50	19.3	--	--	3.39	2.32	33	14	--	--	4.79	2.68	0.5	15
15	11 USG 9-15	38.1	16.11	2.72	36.4	9.25	8.48	--	2.35	35.8	9.60	9.07	--	7.68	6.62	--	--	--	--	--	--	0.65	40
16	18 USG 6-12	41.7	15.5	2.67	46	17.6	15.1	1.01	1.10	47	16.6	18.3	0.931	2.53	1.47	40	13.3	21.5	0.889	4.01	1.90	0.5	9

Plasticity Chart No.	Code No.	Liquid Limit	Plasticity Index	Specific Gravity	UNCONFINED					15 p.s.i. LATERAL PRESSURE						30 p.s.i. LATERAL PRESSURE						'C' Cohesion Kgm/cm²	Angle of Internal Friction
					Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Deviator Str. Kgm/cm²	Degree Saturation %	% Moisture Start	% Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm²	Deviator Stress Kgm/cm²	Degree Saturation %	Percent Moisture Start	Percent Moisture End	Void Ratio Start	Max. Major Principal Stress Kgm/cm²	Deviator Stress Kgm/cm²		
17	2 USG 9-15	38.2	16.6	2.71	49.0	11.3	10.2	0.617	2.41	39.0	11.3	13.8	0.525	4.10	3.04	36	12.0	17.8	0.840	4.96	2.85	0.97	8
19	19 USG 9-15	37.7	16.7	2.69	19.1	7.7	6.8	1.08	1.50	19.0	7.8	6.5	1.10	7.06	6.00	21.8	8.7	8.8	1.08	10.37	8.26	0.40	42
20	19 USG 0-6	37.6	16.5	2.67	25	9.2	--	0.966	1.83	25	9.0	--	0.949	4.57	3.51	24	8.7	--	0.943	5.50	3.39	0.58	26
22	13 USG 9-15	35.0	15.0	2.65	47	14.2	14.4	0.802	1.62	45	14.6	15.3	0.878	3.08	2.02	43	13.7	16.1	0.860	4.64	2.53	0.70	18
23	8 USG 9-15	32.7	15.5	2.59	36.0	12.5	12.6	0.825	0.90	30	10.9	10.8	0.980	6.18	5.12	35.7	13.1	13.2	0.944	10.21	8.10	0.40	41
24	12 USG 6-12	33.4	14.6	2.71	37.8	10.5	9.7	--	0.62	36.4	9.6	9.3	--	6.46	5.40	36.7	9.1	8.9	--	10.03	7.92	0.30	41
25	11 USG 0-6	33.8	13.2	2.62	19.3	7.9	9.38	--	1.15	24.6	8.7	8.7	--	6.26	5.20	--	--	---	--	--	--	--	--
26	16 USG 6-12	31.2	13.3	2.69	32	7.8	8.0	0.648	1.63	33	8.6	9.3	0.692	5.60	4.54	27	8.5	11.6	0.845	5.04	2.93	0.3	40
27	1 USG 9-15	34.7	12.0	2.66	33.7	13.6	13.1	1.05	0.65	35.7	13.0	13.5	0.958	9.31	7.20	35	13.0	13.5	0.958	9.31	7.20	0.2	39
28	16 USG 0-6	36.2	10.2	2.69	35	11.3	--	0.870	1.42	47	11.0	14.0	0.687	3.61	2.55	58	12.3	16.3	0.756	4.75	2.64	0.52	19

TRIAXIAL TEST DATA SUMMARY

SOILS TESTED-- Lethbridge

Consolidation Test Data Summary

Soils Tested Lethbridge

	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
Increasing	17 USG 9-15	1.00	97.1	2.69	43.1	16.1	33.2	36.9		0.035	0.070	0.141	0.282	0.563	1.12	2.25			
										38.7	38.7	38.2	37.4	36.3	34.3	33.2			
	2 USG 9-15	0.644	113	2.69	41.7	9.9	22.1	23.3		0.043	0.070	0.156	0.300	0.585	1.16	2.29	0.043		
										24.6	24.6	24.4	24.0	23.4	22.6	20.8	22.1		
	18 USG 9-15	0.932	99.5	2.67	44.8	15.6	27.6	36.4		0.035	0.141	0.282	0.563	1.12	2.25	4.50	0.035		
										36.6	35.8	34.8	33.7	31.9	28.6	24.0	27.6		
	3 USG 0-6	0.710	109	2.69	44.5	11.9	23.4	28.5		0.043	0.070	0.156	0.300	0.585	1.16	2.29			
										29.9	29.9	29.6	29.2	28.1	26.3	23.4			
Plasticity	(12-18) 10 USG 9-15	0.897	98.7	2.68	38.9	13.2	27.3	34.9		0.043	0.070	0.150	0.300	0.585	1.16	2.29			
										36.8	36.7	36.5	35.8	33.8	31.0	27.3			
	17 USG 0-6	0.513	122	2.65	60.8	11.8	20.1	21.6		0.035	0.123	0.282	0.563	1.12	2.25	4.50	0.035		
										22.4	22.3	21.7	21.0	20.3	19.3	18.0	20.1		
	10 USG 0-6	1.03	92.5	2.68	32.3	12.4	29.0	33.0		0.035	0.070	0.141	0.282	0.563	1.13	2.25	0.035		
										34.3	34.2	33.9	33.2	32.4	30.8	27.0	29.0		
Decreasing	9 USG 9-15	0.720	105	2.70	23.2	6.2	19.7	21.5		0.043	0.070	0.150	0.300	0.585	1.16	2.29	0.043		
										21.6	21.5	21.4	21.0	20.5	19.7	18.2	19.7		
	8 USG 0-6	1.11	92.2	2.65	43.3	18.2	38.1	41.8		0.035	0.070	0.141	0.282	0.563	1.13	2.25	0.035		
										42.9	42.8	42.5	41.9	41.0	39.4	35.8	38.1		
	6 USG 0-6	0.762	114	2.71	67.8	19.1	26.8	30.9		0.035	0.070	0.141	0.282	0.563	1.13	2.25			
										31.9	31.9	31.8	31.3	30.5	29.2	26.8			
	(12-18) 12 USG 9-15	0.897	97.7	2.70	28.0	9.2	25.5	32.3		0.043	0.070	0.156	0.300	0.585	1.16	2.29			
										32.8	32.8	32.3	32.0	31.0	29.0	25.5			

Consolidation Test Data Summary

Soils Tested Lethbridge

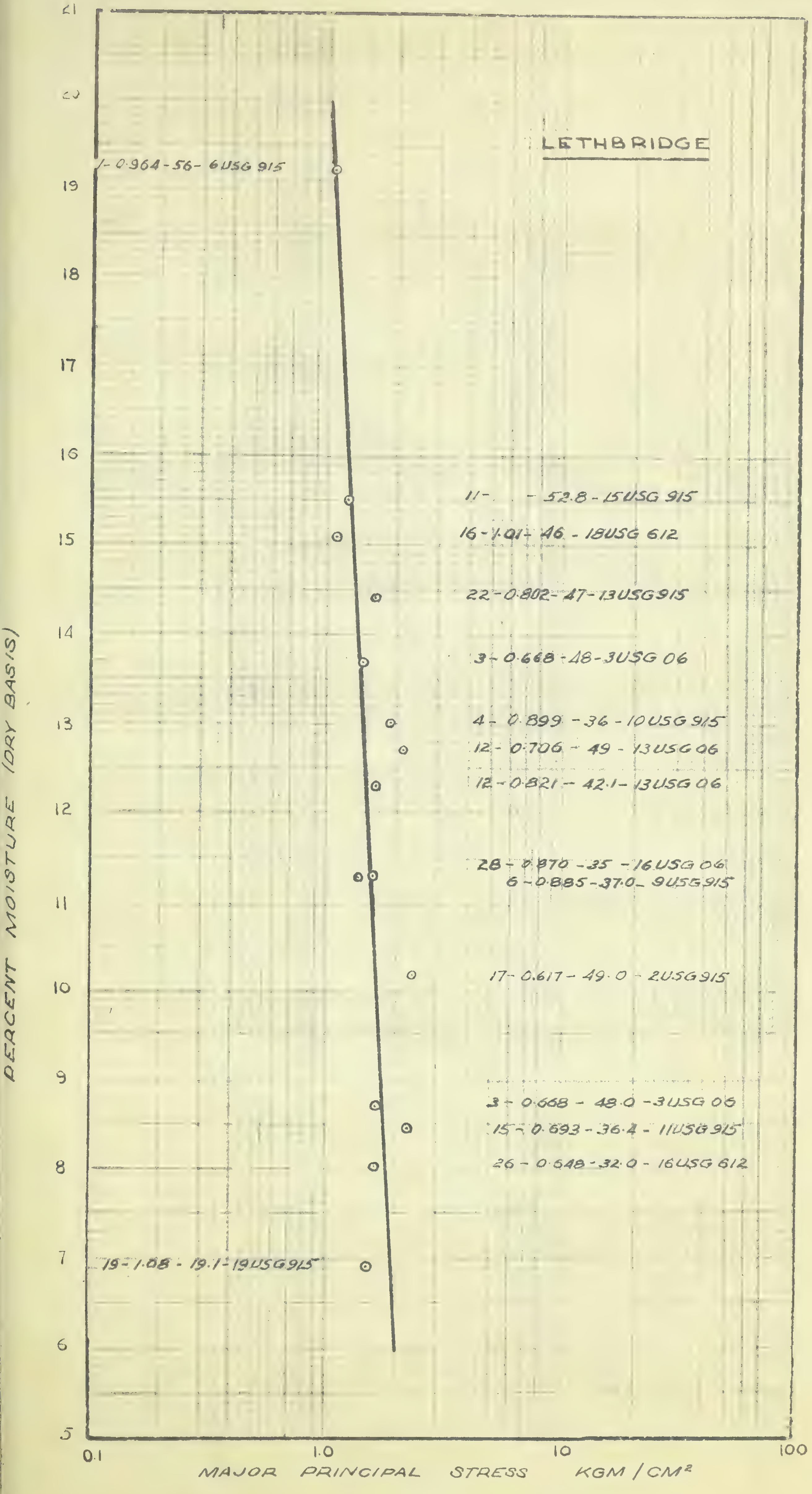
	Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)							
↑	2 USG 0-6	0.532	122	2.67	61.2	12.2	20.9	21.0		0.035	0.070	0.141	0.282	0.563	1.13	2.25	
										22.8	22.8	22.7	22.4	22.0	21.5	20.9	
Increasing	12 USG 0-6	0.686	112	2.68	52.4	13.4	22.9	24.1	2 Samples	0.035	0.141	0.282	0.563	1.13	2.25	4.50	0.035
										26.2	25.9	25.6	25.2	24.7	23.4	20.8	22.9
	13 USG 0-6	0.609		2.67	56.3	12.7	20.6	22.0		0.035	0.070	0.141	0.282	0.563	1.13	2.25	
										22.8	22.7	22.6	22.2	21.9	21.3	20.6	
	18 USG 0-6	0.697	112	2.67	56.6	14.8	22.8	24.3		0.035	0.070	0.141	0.282	0.563	1.13	2.25	0.035
										24.8	24.8	24.6	24.3	23.8	23.2	22.5	22.8
Plasticity	19 USG 0-6	0.850	98.6	2.67	28.5	9.0	26.3	30.2		0.035	0.070	0.141	0.282	0.563	1.12	2.25	0.035
										31.9	31.8	31.5	31.0	30.2	28.6	24.3	26.3
	11 USG 9-15	0.729	106	2.70	32.1	8.7	22.0	27.9		0.043	0.070	0.156	0.300	0.585	1.16	2.29	
										29.1	29.1	28.9	28.4	27.6	25.3	22.0	
	12 USG 0-6	0.590	117	2.70	47.0	10.3	21.6	23.0		0.035	0.070	0.141	0.282	0.563	1.12	2.25	0.035
										25.3	25.3	25.3	24.9	24.5	23.8	22.9	21.6
	11 USG 0-6	0.762	103	2.68	30.7	8.7	30.7	27.9		0.035	0.070	0.141	0.282	0.563	1.13	2.25	0.035
										29.8	29.8	29.5	28.9	28.0	26.8	24.5	20.8
Decreasing	(12-18) 19 USG 9-15	1.00	91.4	2.68	19.4	7.3	23.2	39.4		0.035	0.070	0.141	0.282	0.563	1.12	2.25	4.50
										41.0	40.9	40.5	39.7	38.0	33.6	27.9	23.2
	(6-12) 13 USG 9-15	1.31	83.6	2.68	79.7	15.3	34.8	40.9		0.035	0.070	0.141	0.282	0.563	1.13	2.25	0.035
↓										41.3	41.1	40.4	38.3	37.6	35.1	31.0	34.8

Consolidation Test Data Summary

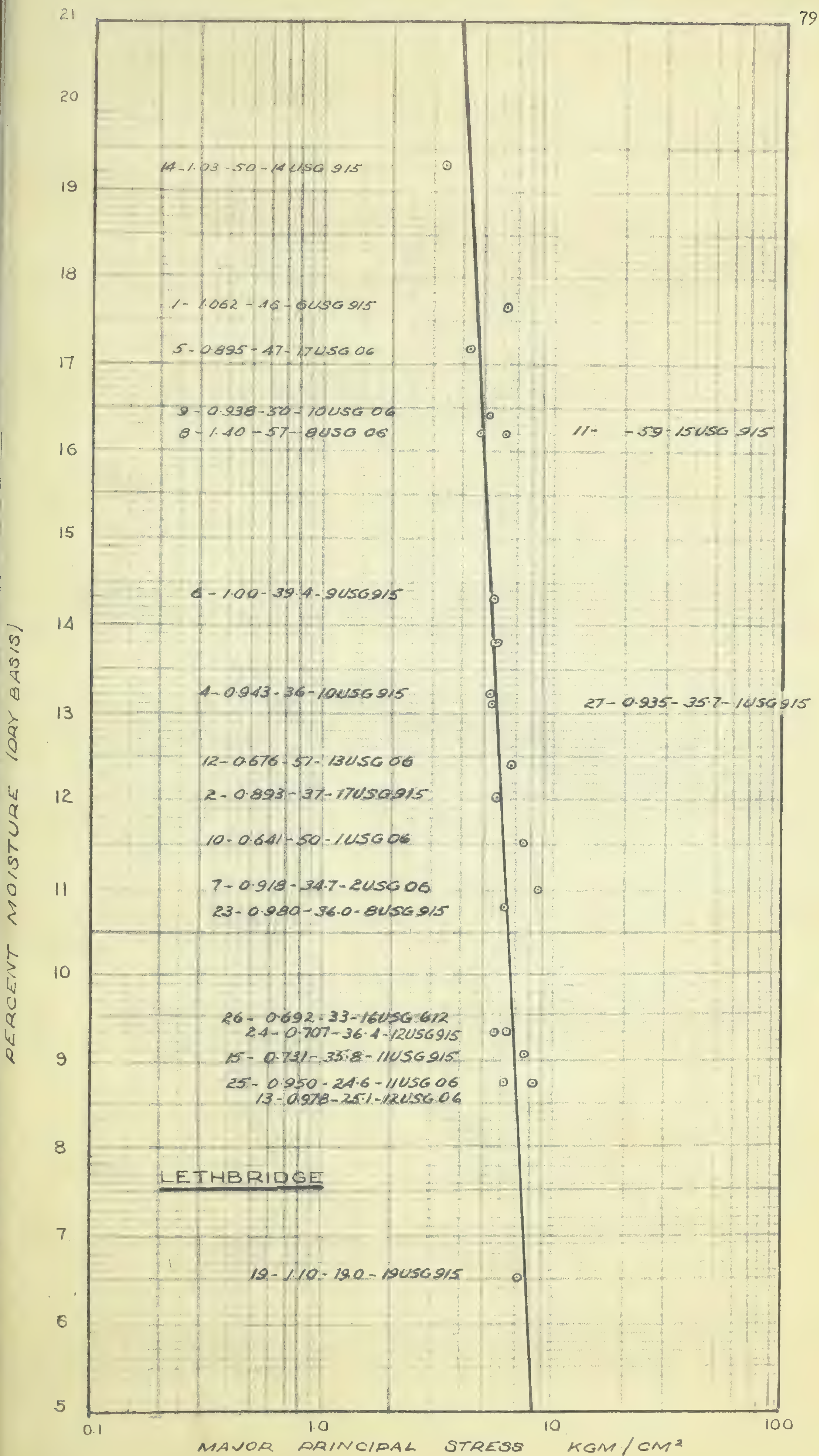
Soils Tested Lethbridge

Sample Code Number	Void Ratio Start + Measurement	Unit Weight Start lbs. per ft.^{3}	Specific Gravity	Degree Saturation Start Percent	% Moisture start of test	% Moisture end of test	% Moisture saturated start (computed)	Comments etc.	Pressure on soil kg/cm^2 (values on top line) / Percent Moisture Assuming Saturation (values on bottom line)									
(12-18) 8 USG 9-15	0.973	96.1	2.67	38.2	13.9	31.8	39.9		0.043	0.070	0.156	0.300	0.585	1.16	2.29			
									42.7	42.7	41.3	40.7	39.1	36.0	31.8			
16 USG 0-6	0.758	--	2.66	36.4	10.4	24.6	27.4		0.035	0.070	0.141	0.282	0.563	1.12	2.25	0.035		
									28.1	28.0	27.8	27.5	26.7	25.6	23.6	24.6		
1 USG 9-15	--	87.1	2.67	35.8	16.6	31.1	40.6		0.035	0.070	0.141	0.282	0.563	1.13	2.25			
									40.7	40.5	39.8	38.7	36.9	34.7	31.1			
14 USG 0-6	0.527	120.3	2.69	56.5	--	19.1	19.6		0.035	0.141	0.282	0.563	1.13	2.25	4.50	0.035		
									20.3	20.1	19.8	19.5	19.0	18.5	17.5	19.1		

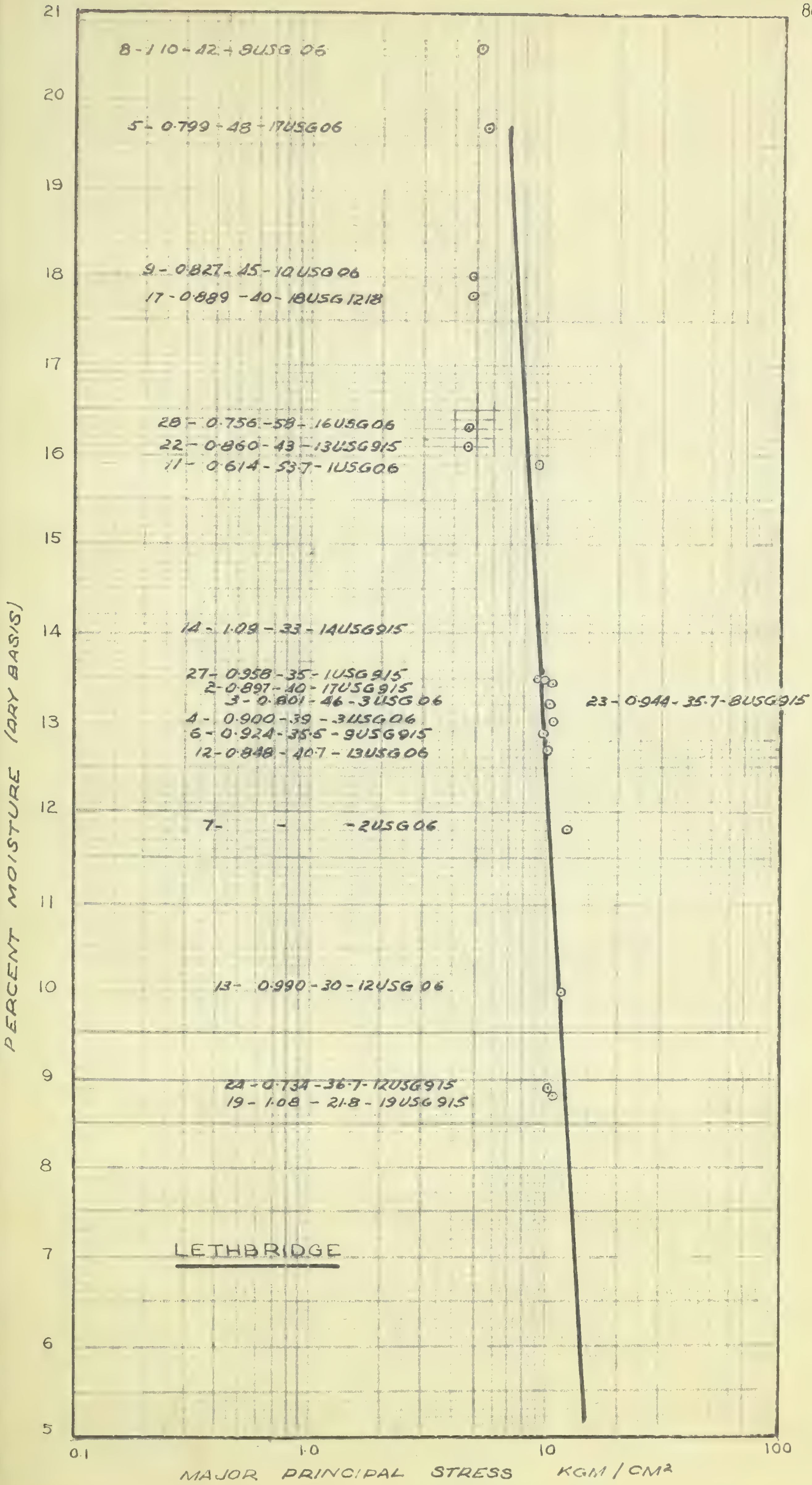
← Increasing Plasticity Decreasing →



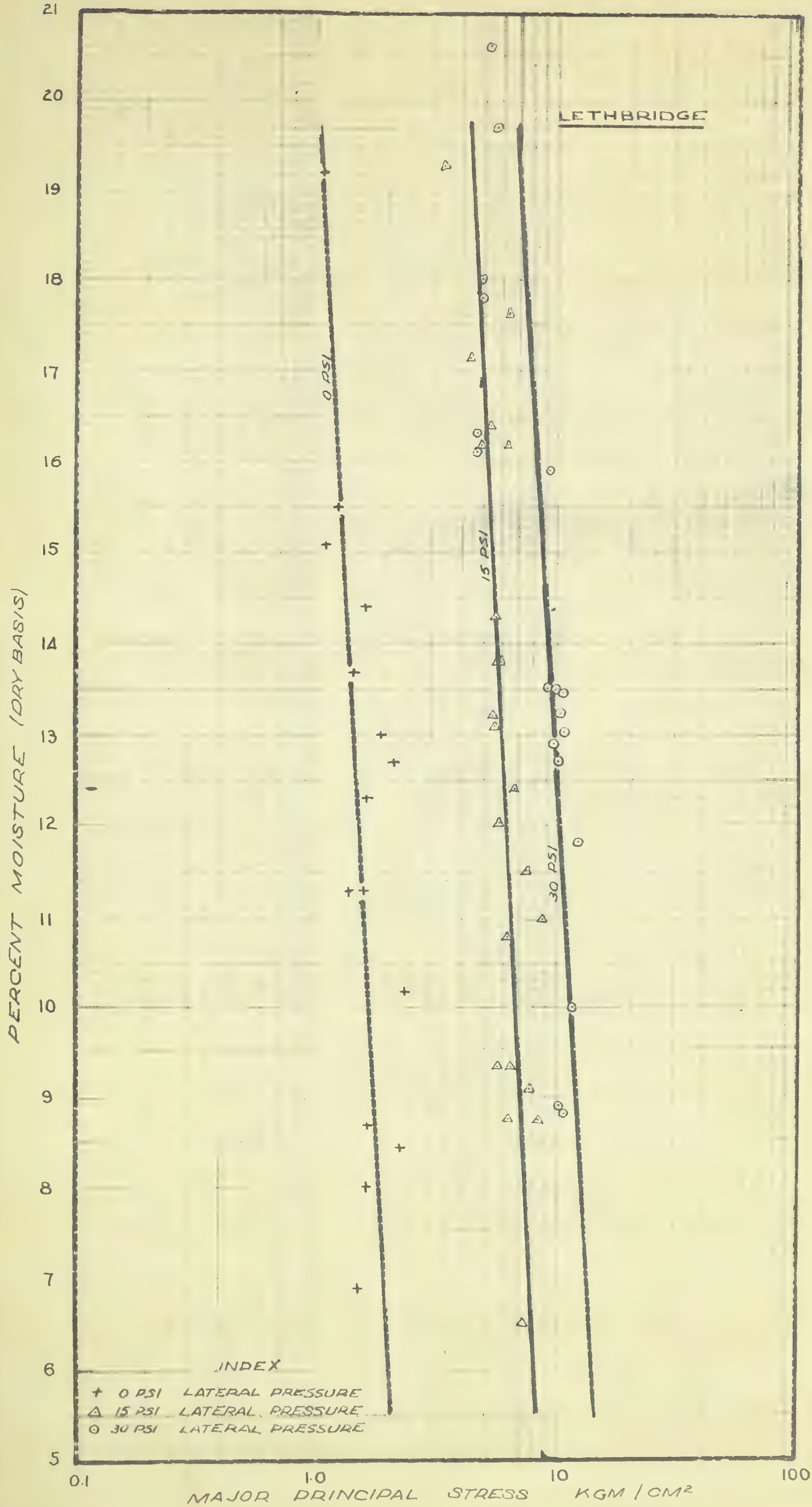
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 0 - LATERAL PRESSURE



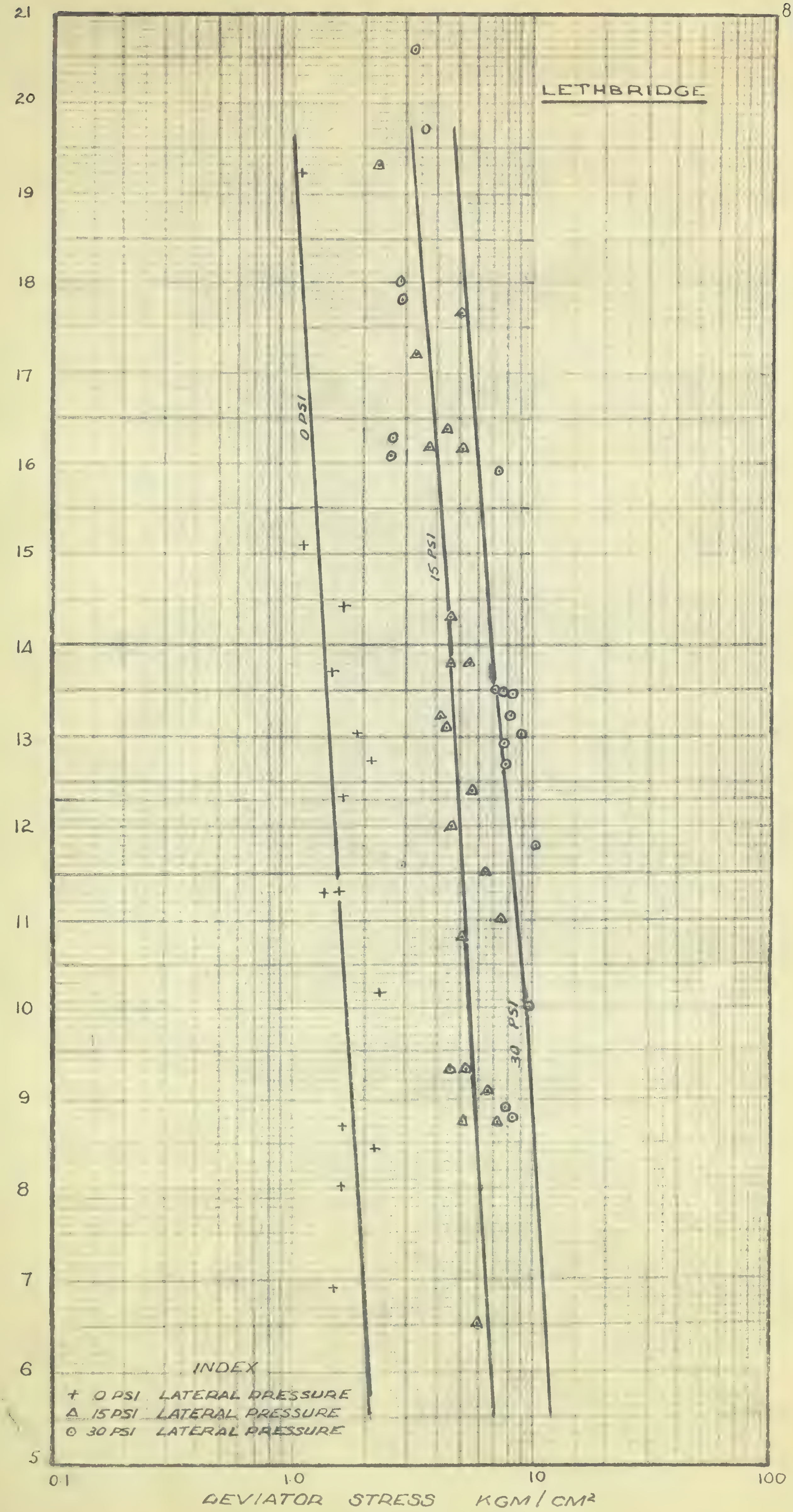
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS - 15 PSI LATERAL PRESSURE



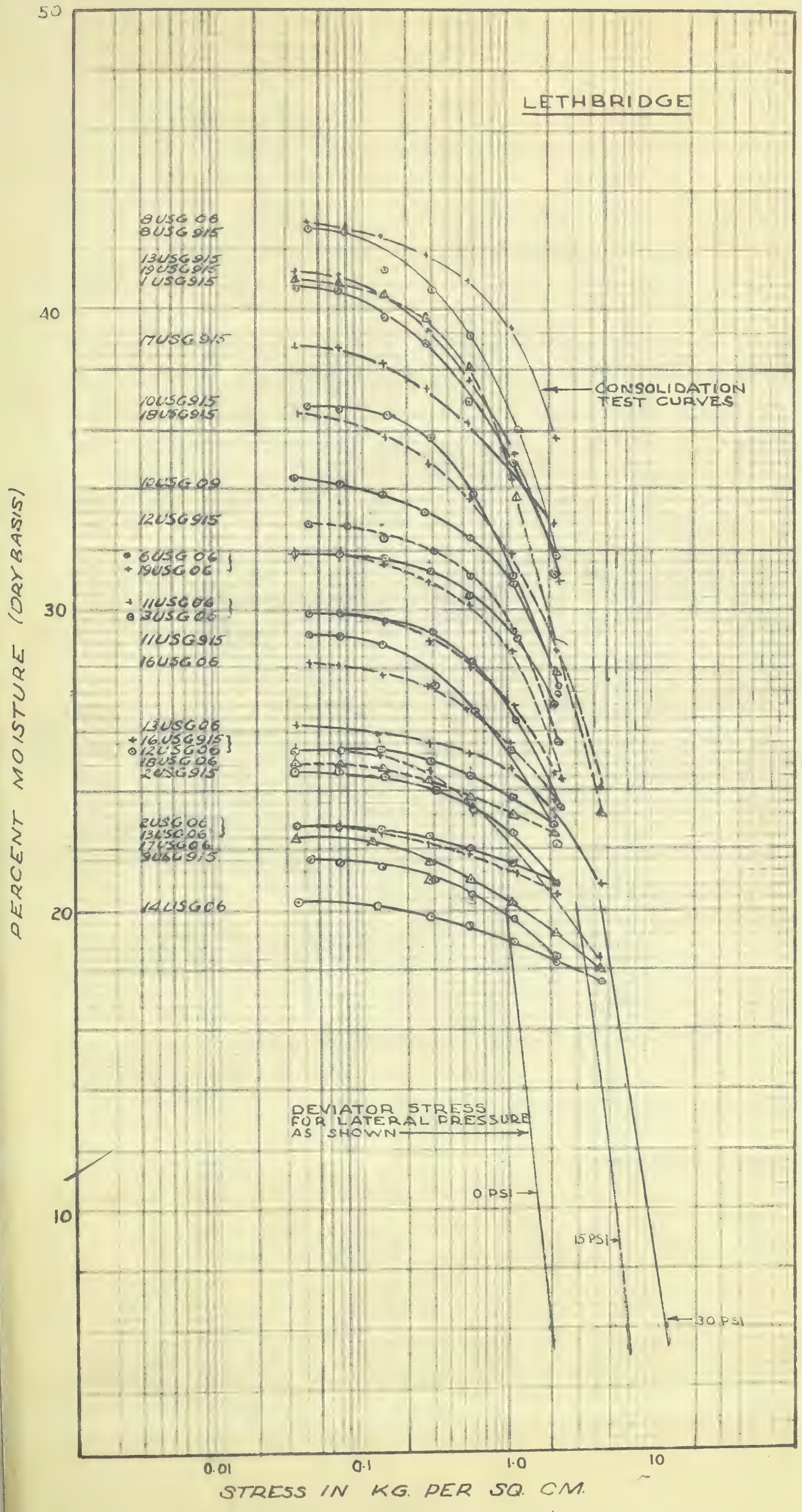
PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS. MAJOR PRINCIPAL STRESS - 30 PSI LATERAL PRESSURE



PERCENT MOISTURE (DRY BASIS) AT END OF TEST
VS MAJOR PRINCIPAL STRESS



PERCENT MOISTURE (DRY BASIS) AT END OF TEST VS. DEVIATOR STRESS



CONSOLIDATION AND TRIAXIAL TEST
MOISTURE - STRESS RELATIONS

12. DISCUSSION OF RESULTS

The results have been discussed first for each airport separately.

A. Saskatoon --The Saskatoon samples were found to be the most similar to the Massena and Chicago clays reported by Rutledge. A high degree of saturation existed for most of the Saskatoon samples, averaging approximately 86.2 % but ranging from 52% to 100%. The strength-moisture plots give better relationships at the higher lateral pressures and this has been shown to be partly due to decreased probable errors at the higher pressures.

Triaxial test strengths may have been actually higher than indicated. This has been due to load increments that were too large, for example, loads of 10, 20, then failure at 50gms. on the pan for unconfined samples. In this case the indicated compressive strength may be as small as 40% of the actual value.

At higher lateral pressures the load increment was a smaller per cent of the total load and less probable error resulted. The confined tests therefore gave the better graphs and were more suitable for studying the effects of differences in plasticity, void ratio and degree of saturation.

The plasticity chart indicates that the cohesive soils fall practically in one group and not much difference should be expected due to differences in plasticity. This was actually the case, and the main relationship appears between moisture content and strength. Large initial void ratios tended to give smaller compressive strengths for the same moisture content but there were a number of obvious exceptions. These may be due to some samples breaking prematurely or possibly for the existence of an optimum degree of saturation for a given moisture content. Insufficient data does not permit any substantial verification of the latter.

The plot of moisture content versus $\sigma_1 - \sigma_3$ at failure gave one best 'fit curve' but with considerably more deviation than in the results reported by Rutledge. Differences in degrees of saturation and plasticity probably are the main causes. Most of the consolidation curves were very similar to recompression branches obtained in reloading consolidation samples. (Fig. 7).

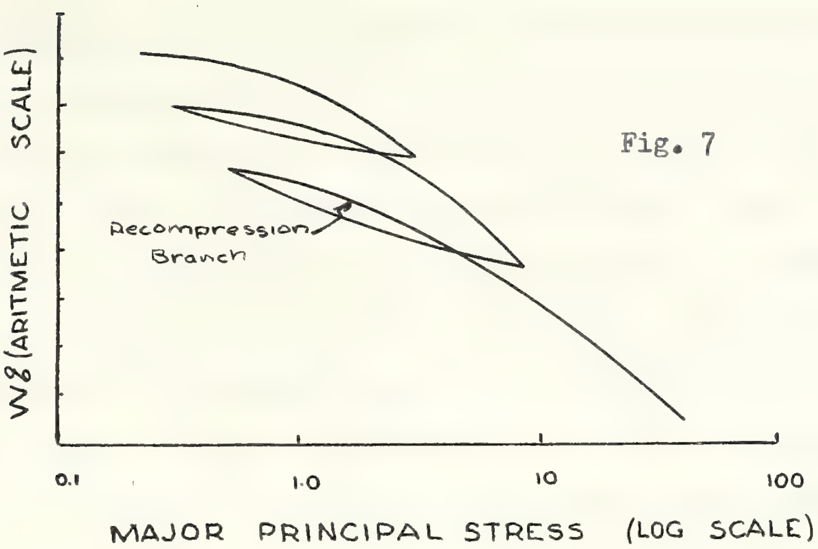


Fig. 7

This would indicate that the samples were considerably pre-consolidated. It appears also that the pre-consolidation may have been greater than that obtained by the standard graphical method. (Fig. 8).

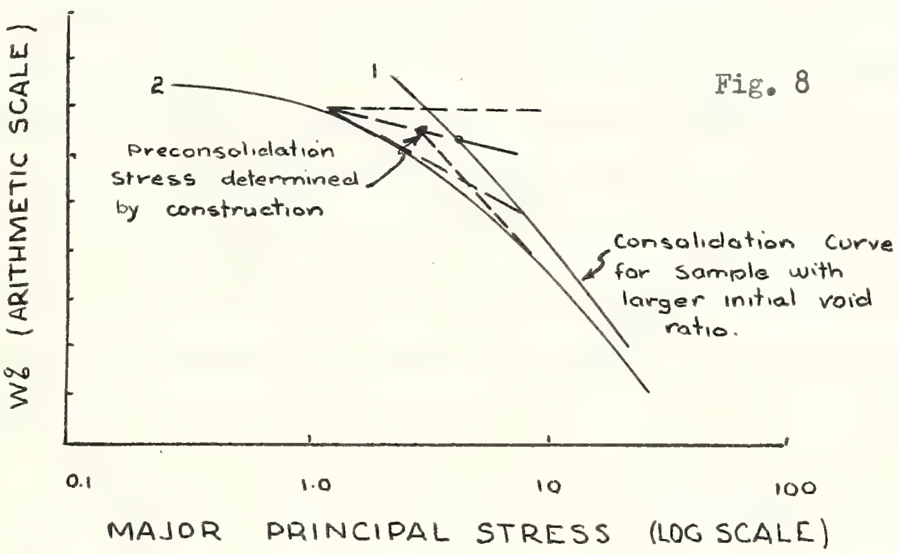


Fig. 8

If initially high void ratios are considered to be due to less pre-consolidation, then curve '2' is not on the virgin compression curve until it becomes tangent to curve '1'. There is no conclusive evidence, however, to prove that the differences in curves '1' and '2' are due only to differences in pre-consolidation. Differences in soil types, effect of intergranular bonds and other factors may cause the differences. They also may have been deposited originally with varying void ratios.

The establishment of the two curves in Casagrande's hypothesis requires the determination of the virgin compression consolidation curve.

Soils from airport sites have generally been subjected to shrinkage and other pressures and were therefore considerably consolidated at some time. The difficulty which arises in determining the virgin compression curve in the lower pressure range is not easily overcome unless none pre-consolidated samples are available or extrapolation is used. Extrapolation, of course, cannot give very accurate results. It appears that the triaxial test strengths may not be influenced to a great extent by pre-consolidation. If this can be proven for a given soil, curve 'A' of the hypothesis may be determined by using the procedures outlined in S , P-I, in reverse with the information on curve 'B'. Such a method would be very useful. Curves 'A' and 'B' could be established using a number of Q_c tests, no consolidation tests would be necessary, the virgin compression curve would be obtained and 'S' test results obtained without performing any 'S' tests. The procedure would be limited to saturated homogeneous soils only.

B. Grande Prairie--The plasticity chart indicates a wide range of cohesive soils but the differences in plasticity do not appear to be important governing factors. The plasticity numbers do not fall in any logical order or grouping. Large initial void ratios appear to correspond to smaller compressive strengths but there are a number of exceptions. Although the degree of saturation is determined by the per cent moisture and void ratio it appears to be another independent factor influencing strengths.

The graphs of W% versus major principal stress at failure (semi logarithmic) gave straight lines for each lateral pressure in the moisture content ranges studied. Initial moisture contents were plotted for samples 22U - SG - 9-15, 2U - SG - 9-15 and 1U - SG - 10-16, since final values were not available. It was found in the other samples that the final moisture content was only slightly less than the initial W%. The error that was introduced would be small and the points were important in establishing the best 'fit lines'.

The degree of saturation was less than for the Saskatoon soils. Deviator stresses at failure gave separate best 'fit lines' for each lateral pressure. Higher deviator stresses at failure corresponded to the larger lateral pressures.

and other pressures and were therefore considerably consolidated at [illegible]

[illegible] curve in the lower pressure range is not easily [illegible] [illegible] or extrapolation [illegible]. [illegible], of course, cannot give very accurate results. [illegible] the triaxial test strength may not be [illegible] [illegible]. It [illegible] for a [illegible] soil, [illegible] using [illegible]

[illegible] and [illegible] no consolida-tion tests [illegible], the [illegible] compression curve could be obtained [illegible] results obtained without performing any [illegible] tests. The [illegible] would be limited to saturated homogeneous soils only.

[illegible] index [illegible] [illegible] in many [illegible] cohesive soils [illegible] difference in plasticity [illegible] [illegible] [illegible] [illegible] plasticity appears to [illegible] [illegible]. [illegible] ratio appears to [illegible] [illegible] strength [illegible] there are a number of exceptions. [illegible] of saturation is determined by [illegible] and void ratio it appears to be a better index [illegible]

The graphs of [illegible] major principal stress at failure (semi logarithmic) [illegible] for each lateral pressure in the moisture content [illegible] studied. Initial moisture contents were plotted for samples [illegible] - 9-15 and [illegible]
since final values were not available. [illegible] the other [illegible]

the initial [illegible]. The error that was introduced would be small and [illegible] in establishing the best fit lines.

The degree of saturation was less than [illegible] the [illegible]
[illegible] deviator stresses at failure gave separate best fit lines
[illegible] lateral pressures. Higher deviator stresses at failure [illegible]

The consolidation curves indicated considerable pre-consolidation. Because of the different degrees of saturation, the triaxial test and consolidation test curves overlapped very little.

C. Fort St. John--Graphs for the Fort St. John soils were very similar to those from Grande Prairie. Void ratio had a less marked influence on the compressive strength. The main factor influencing strength appears to be the final moisture content.

D. Lethbridge--The Lethbridge soils were considerably less plastic and less saturated than the Saskatoon, Grande Prairie or Fort St. John soils. Decreased strengths with increased void ratios are very apparent and the increase in maximum deviator stress with increased lateral pressures is larger. The maximum deviator stresses at failure plot completely below the consolidation curves and comparison is not possible. Large pre-consolidation pressure are indicated.

13. VARIABLES AFFECTING STRENGTH

The results have been discussed here under the headings given in S-8, P-I, Verification was obtained on the effect of most of these variables.

A. Soil Types--The differences in soil types were shown on the plasticity chart. The plasticity did not make an appreciable difference in the strength of the cohesive soils from a given site. The soils with very high plasticities appear to have somewhat greater compressive strengths.

B. Condition of Soil--No accurate records were made describing the initial undisturbed condition of the soil--e.g., fill material, stratification, etc. These factors probably would have explained many of the large deviations in the results.

C. Stress History of the Soil--Much larger pre-consolidation pressures than have been indicated by the construction method were probably had. (See discussion under Saskatoon soils, S-12A, P-II).

D. Type of Test--Only standard consolidation and 'Q'triaxial tests were run. Conclusions are therefore limited to these two tests.

E. Minor Principal Stress--The effect of the minor principal

stress in the triaxial test becomes more evident with lesser degrees of saturation. The values of $\sigma_1 - \sigma_3$ at failure are displaced in the direction of increased strengths for increased lateral pressures. This displacement was greater for the lesser degrees of saturation ranging from a single line in the highly saturated Saskatoon soils to three considerably separated lines for the Lethbridge soils.

F. Initial Water Contents--For 'Q' tests very little difference was noted between end and initial water contents. Theoretically there should be no difference.

H. Initial Degree of Saturation--This is closely related with initial moisture content and initial void ratio but appears to be for some soils an independent factor influencing strength, (e.g., Grande Prairie).

I. Final Moisture Content--The final moisture content appears to be the main influencing factor of strength for the more saturated soils. Void ratio and degree of saturation also have a marked effect for the less saturated soils.

J. Degree of Saturation and Void Ratio at End of Test--These quantities were not measured in the D.O.T. program and their effect cannot be ascertained.

14. CONCLUSIONS

A. The Casagrande working hypothesis is limited to saturated homogeneous clays for which a virgin compression curve can be determined.

B. No single relationship of compressive strength (deviator stress at failure) versus moisture content, exists for partially saturated soils in 'Q' tests. Different curves result for different lateral pressures. Increased dry density (smaller void ratios) at start appear also to increase the strengths.

C. For a given lateral pressure the best 'fit line' on the major principal stress versus the per cent moisture at the end of test (triaxial test), semi logarithmic plot, is very nearly a straight line for the soils and moisture contents investigated.

D. Virgin compression curves for the lower pressures are difficult to establish for airport subgrades due to the large shrinkage

[illegible]

[illegible] in the highly saturated [illegible] considerably [illegible] than for the [illegible]

7. [illegible]

[illegible]

[illegible] to be the main influencing factor of strength for the more saturated soils. [illegible] degree of saturation [illegible] a major effect for the less saturated soils.

9. [illegible] of [illegible]--These [illegible] not measured in [illegible] effect [illegible]

1. [illegible] is [illegible] to saturation [illegible] which [illegible] curve [illegible]

2. [illegible] relationship [illegible] curves [illegible] soils in [illegible] for different [illegible] increased [illegible] (swelling void ratios) at about [illegible]

and overburden pressure that may have caused pre-consolidation. Pre-consolidation pressures may have been greater than those indicated by the present methods of evaluation.

E. Where samples have very low field moisture contents and triaxial data were obtained at these moisture contents, a comparison of strength-moisture plots is not possible to the consolidation curves. e.g., Lethbridge, Grande Prairie and Fort St. John soils.

F. The two curves of Casagrande's Hypothesis apply to ideal conditions which are approached with soils approaching complete saturation and uniformity.

15. RECOMMENDATIONS FOR FUTURE RESEARCH

A. The failure load increments used in the D.O.T. triaxial tests were too large and resulted in large probable errors.

Compressive strengths should be estimated and large load increments may be used up to say 80% of the estimated value. The final increments should be small enough to assure more accuracy.

B. 'Q', 'Q_c' and 'S' tests should be performed on partially saturated samples with controlled moisture contents.

C. Pore pressures should be measured to ascertain their effect on strength.

D. Accurate calibration of membrane effect on strength for the soils tested should be made.

E. Much higher consolidation pressure should be used to determine the lower portions of the virgin compression curve. The possibility of using curve B, to determine curve A from the converse of the working hypothesis, merits attention. This may be a means of establishing the virgin compression curve for the lower pressure ranges.

F. The strength of soils in this thesis has been approached purely from a mechanical point of view. Pre-consolidation may have been caused by other factors, for example, electrosmotic action, and the suggestion is made that strengths be investigated also from the physical, chemical, etc., viewpoints.[1]

(1) Geotechnics and Geotechnical Research--Edmund F. Preece; Proceedings twenty-seventh annual meeting, 1947, Highway Research Board.

PART III

INVESTIGATION OF CERTAIN INCONSISTENCIES IN RELATIONSHIPS OF SUBGRADE SUPPORT, ANGLE OF INTERNAL FRICTION AND SLOPE FACTOR

1. The Rutledge and McLeod Reports

From the Rutledge report, discussed in Part I, it was shown that the angle of internal friction for a saturated cohesive soil depended on the drainage conditions employed in the triaxial test. No relationship was indicated for compressive strength versus ϕ, nor between ϕ and the final moisture content at the end of the test. For a partially saturated soil, it follows that the angle of internal friction obtained in a 'Q' test depends on the degree of saturation which limits the intergranular stresses that are developed and the value of ϕ which is mobilized before failure. A given degree of saturation thus in effect corresponds to a certain degree of drainage.

Dr. McLeod in his report, "Airport Runway Evaluation in Canada", indicates a relationship between subgrade support and functions of 'C' and ϕ. The subgrade support is defined as the load applied by a circular plate of given diameter on a subgrade which will give a certain deflection after a certain number of repetitions of load application. To correspond to the traffic obtained on Canadian Airports he uses the following:

Diameter of plate	=	30"
Repetitions of load	=	10
Deflection	=	0.2"

The subgrade support is a measure of the bearing strength of a subgrade and good relationship has been obtained between bearing plate data and other types of bearing and penetration tests. (e.g., C.B.R., Housel, and Cone Bearing. The functions of 'C' and ϕ that are used are explained in S-2, P-III.

It appears inconsistent, however, that a relationship should be obtained between subgrade support and ϕ and 'C' when no similar relationship is indicated for compressive strengths and 'ϕ' and 'C'. An attempt is made in Part III of this thesis to analyze these differences.

FIGURE 1

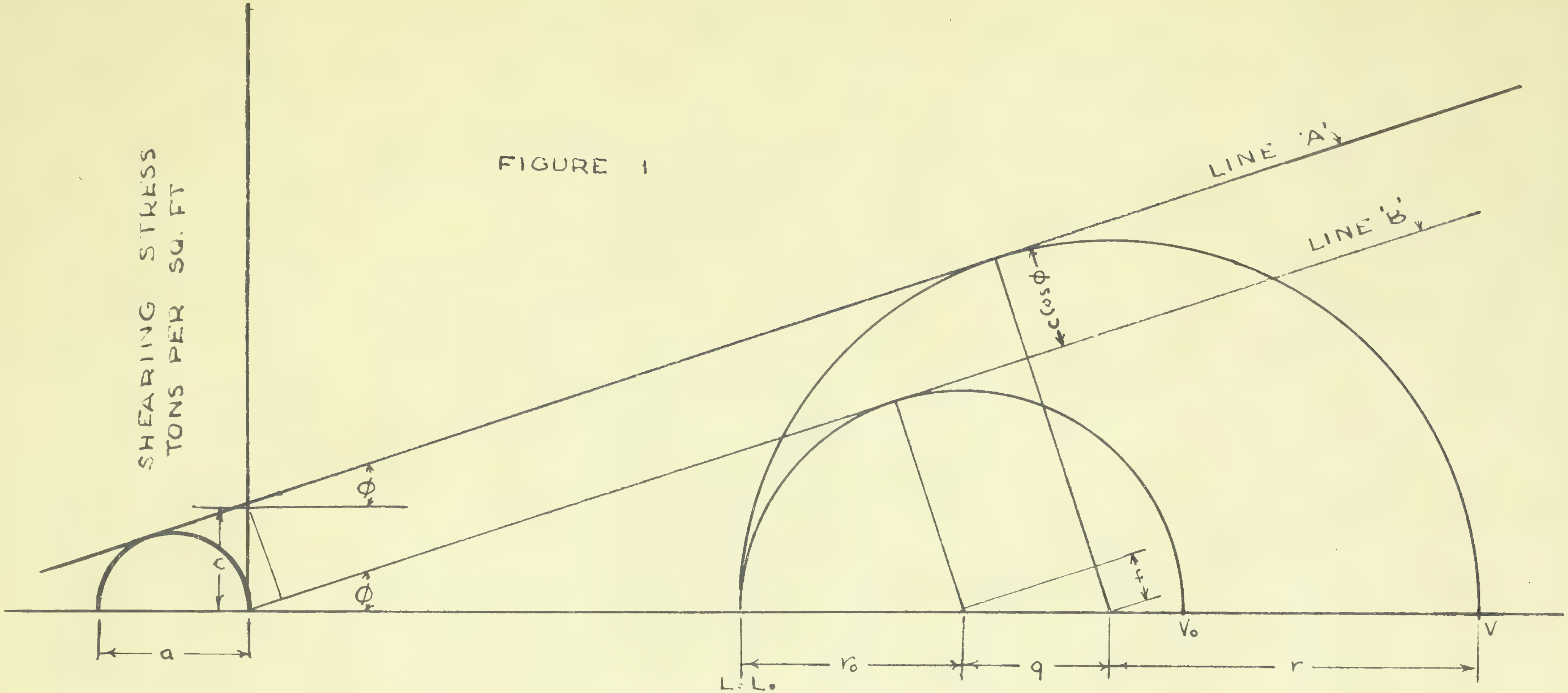


DIAGRAM ILLUSTRATING CERTAIN GEOMETRICAL RELATIONSHIP FOR TRIAXIAL COMPRESSION TEST DATA

2. <u>Functions of 'ϕ' and 'C' Used by Dr. McLeod</u>

Consider Fig. 1, Lines A and B are parallel, the cohesion 'C' equals zero for line B. Both Mohr's circles have the same lateral pressure $L = L_o$ and the subscript 'o' is used with the values obtained from line B. If the geometrical and trigonometrical relationships of the two Mohr's envelopes are analyzed the following equations result:

a. $$\frac{V - L}{V} = 1 - \frac{L_o}{V_o + \frac{2c \cos \phi}{1-\sin\phi}}$$

b. $$r_o = \frac{L_o}{\text{Cosec } \phi - 1}$$

c. $$V_o = L_o + 2r_o$$

d. $$\frac{V_o - L_o}{V_o} = \frac{2 \sin \phi}{1 + \sin \phi}$$

e. $$a = - 2 \text{ C Tan } (45-\phi/2)$$

f. $$q = \frac{c \cos \phi}{1-\sin \phi}$$

g. $$f = q \sin \phi$$

where 'C' = cohesion and 'ϕ' = angle of internal friction.

Dr. McLeod then proceeded to make plots of:

a. $\frac{V-L}{V}$ versus L

b. $\frac{V-L}{V}$ versus V

c. $\log \frac{V-L}{V}$ versus $\log L$

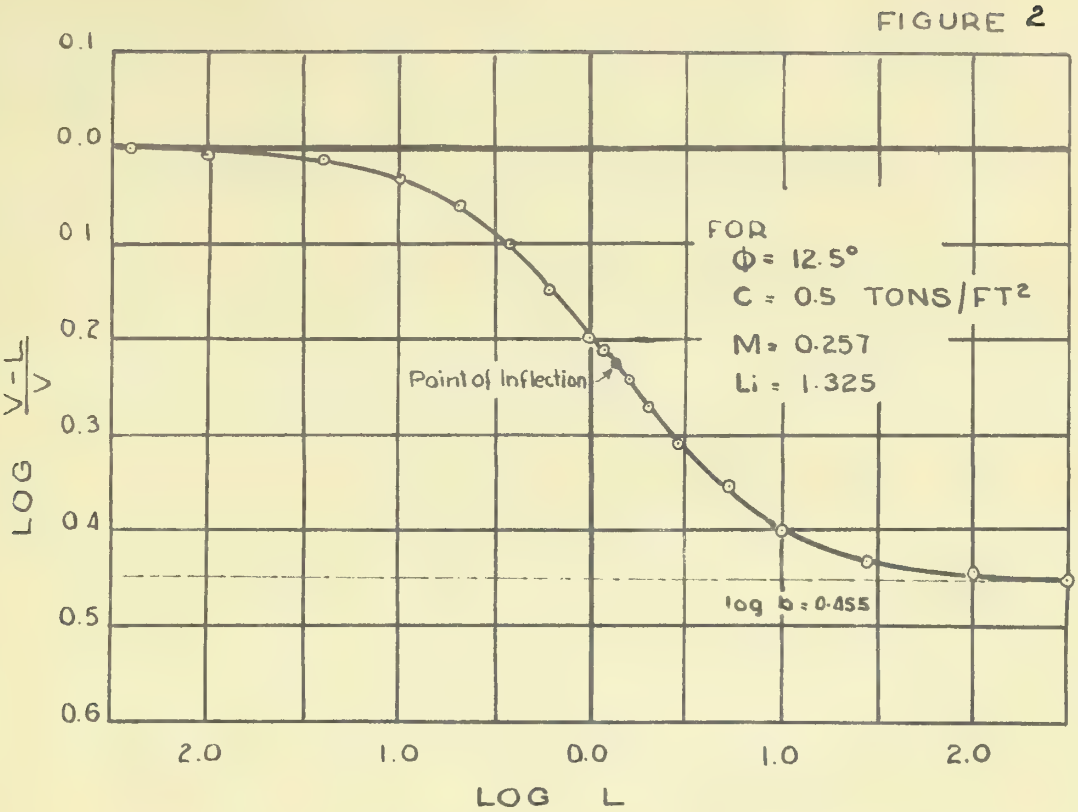
d. $\log \frac{V-L}{V}$ versus $\log V$

There is nothing significant about these plots and they are simply other forms of expressing the Mohr's rupture envelope conditions. The plot of $\frac{V-L}{V}$ versus log L, Fig. 2, has a point of inflection which Dr. McLeod considered might be "a definite quantitative value that would correlate bearing plate and triaxial compression tests". The following relationships are obtained for this point of inflection:

$$\text{Log } \frac{V-L}{V} - \frac{V_o L_o}{V_o} = \text{Log K} - \text{Log}\left\{ L-\left[(-2C \tan 45 - \phi/2)\right]\right\}$$

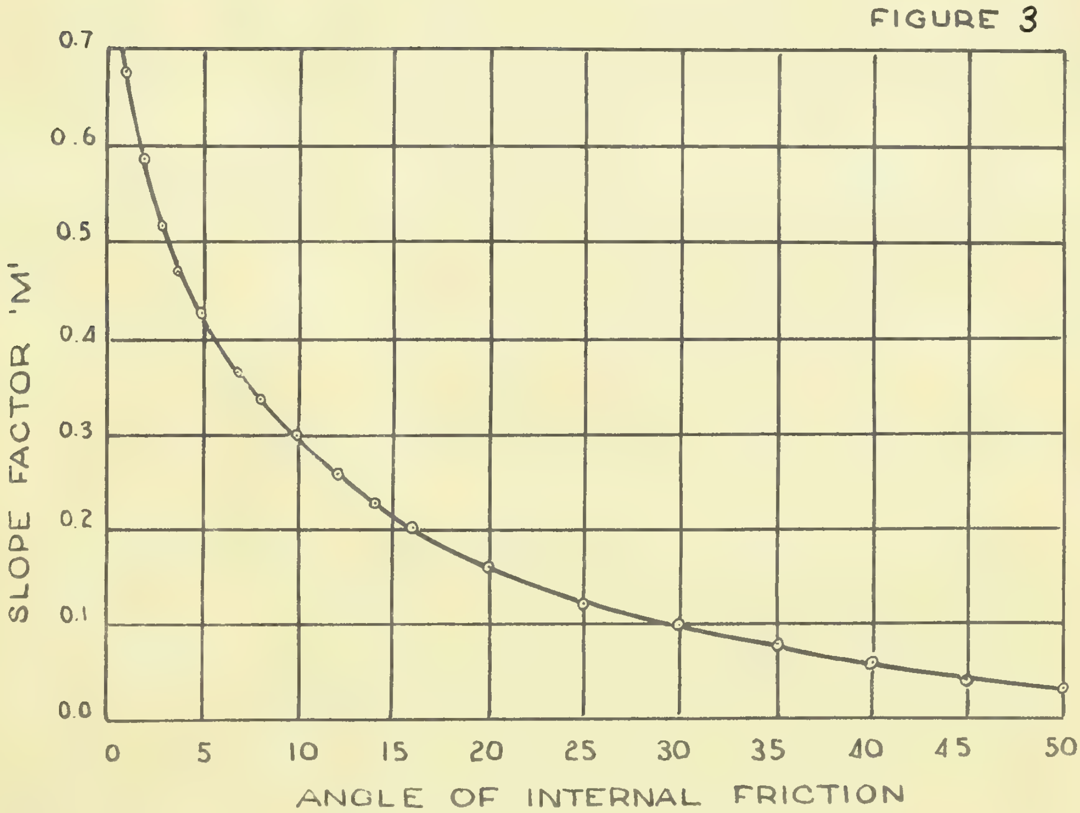
which is the equation of the line.

FIGURE 2



LOG OF RATIO $\frac{V-L}{V}$ VS. LOG L
TRIAXIAL COMPRESSION TEST

FIGURE 3



SLOPE FACTOR 'M' VS. ANGLE OF INTERNAL FRICTION
TRIAXIAL COMPRESSION TEST

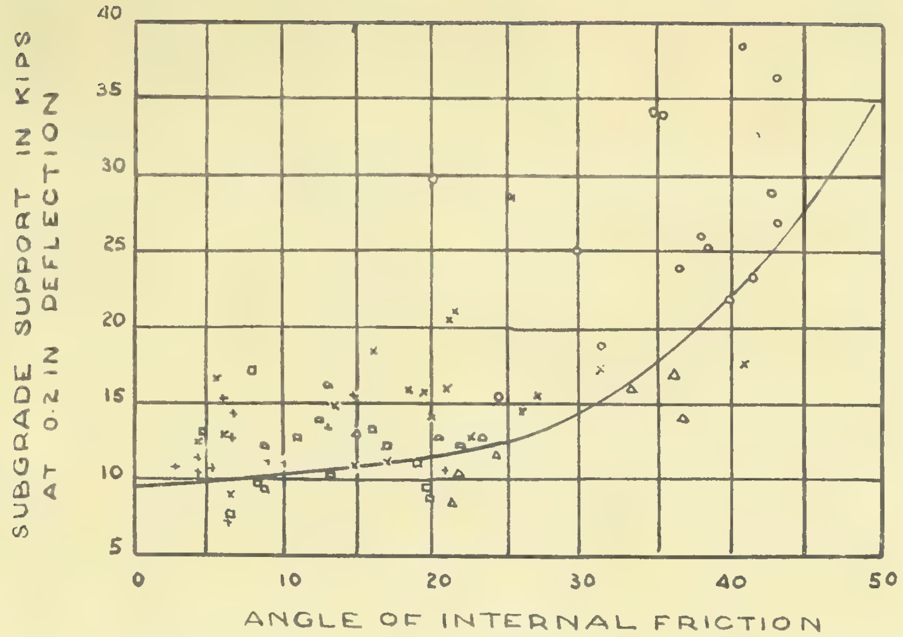


FIGURE 4

SUBGRADE SUPPORT IN KIPS AT 0.2 IN. DEFLECTION
VS. ANGLE OF INTERNAL FRICTION

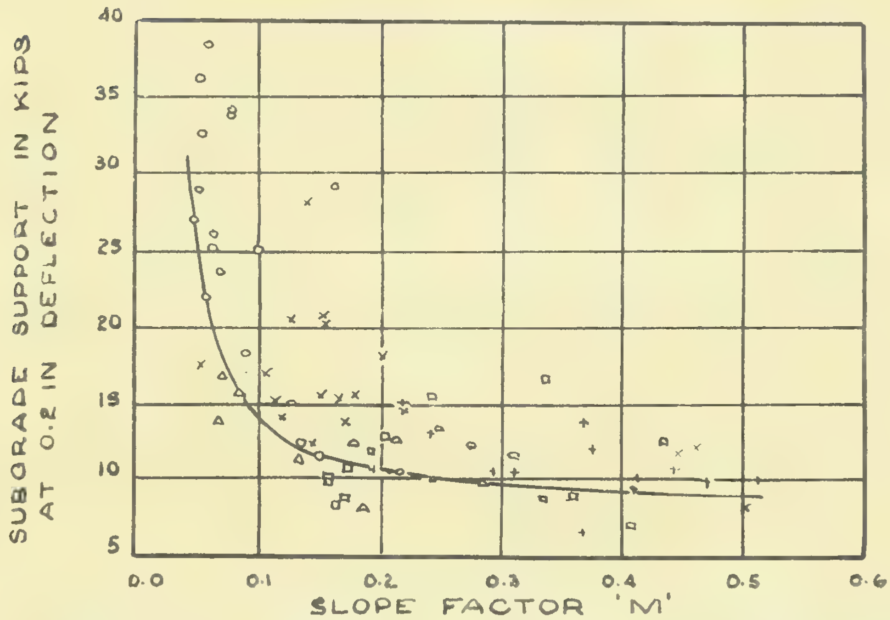


FIGURE 5

SUBGRADE SUPPORT IN KIPS AT 0.2 IN. DEFLECTION
VS. SLOPE FACTOR 'M'

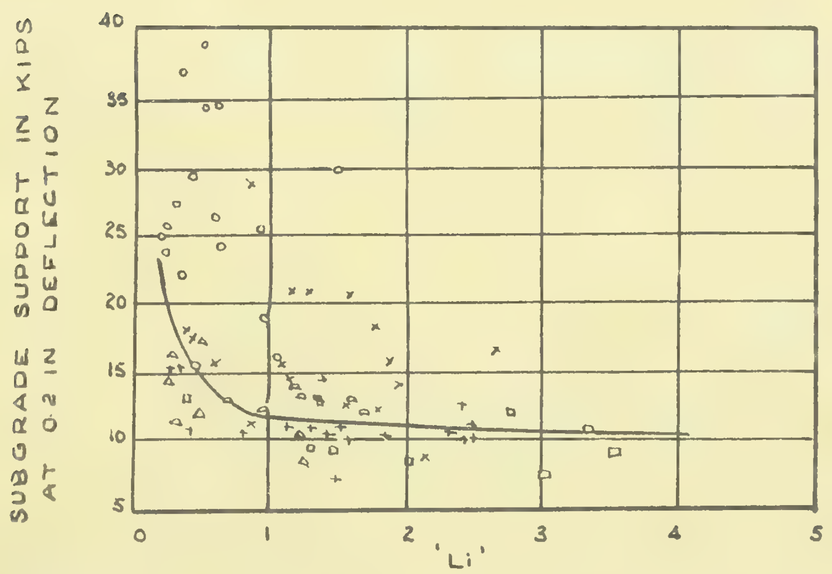


FIGURE 6

SUBGRADE SUPPORT IN KIPS AT 0.2 IN. DEFLECTION
VS 'Li'

NOTE
SUBGRADE SUPPORT
ON 30" DIAM. PLATE AT
0.2" DEFLECTION AFTER
10 REPETITIONS OF LOADING

INDEX

LETHBRIDGE	o
GRANDE PRAIRIE	x
FT. ST. JOHN	□
SAKATOON	+
WINNIPEG	⌂
REGINA	△

The point of inflection is given by the second derivative:

$$\frac{d^2 \log \frac{V-L}{V}}{(d \log L)^2}$$

and occurs at a lateral pressure of:

$$L = L_i = -a\sqrt{1/b}$$

The slope at the point of inflection is given by:

$$d \frac{\log \frac{V-L}{V}}{d \log L} = \text{slope factor 'm'}$$

3. Subgrade Support And Point of Inflection Factors

a. Relationship Depending on 'Ø' only--From the above analysis it may be shown that the slope factor 'm' is a function of 'Ø' only and the relationship is expressed by Fig. 3. Therefore the plot of subgrade support versus the slope factor gives the same relationship as subgrade support versus angle of internal friction (Figs. 4 and 5). Similarly subgrade support versus $\frac{V_i - L_i}{V_i}$, $\frac{V_i}{L_i}$ or $\frac{V_i - L_i}{L_i}$ are functions of subgrade support and 'Ø' only.

b. Relationships Depending on 'C' and 'Ø'--Plots of subgrade support versus L_i or V_i depend on the value of the cohesion 'C' and the angle of internal friction 'Ø'. The plot of subgrade support versus L_i is reproduced in Fig. 6 and subgrade support versus V_i is omitted since no relationship is indicated.

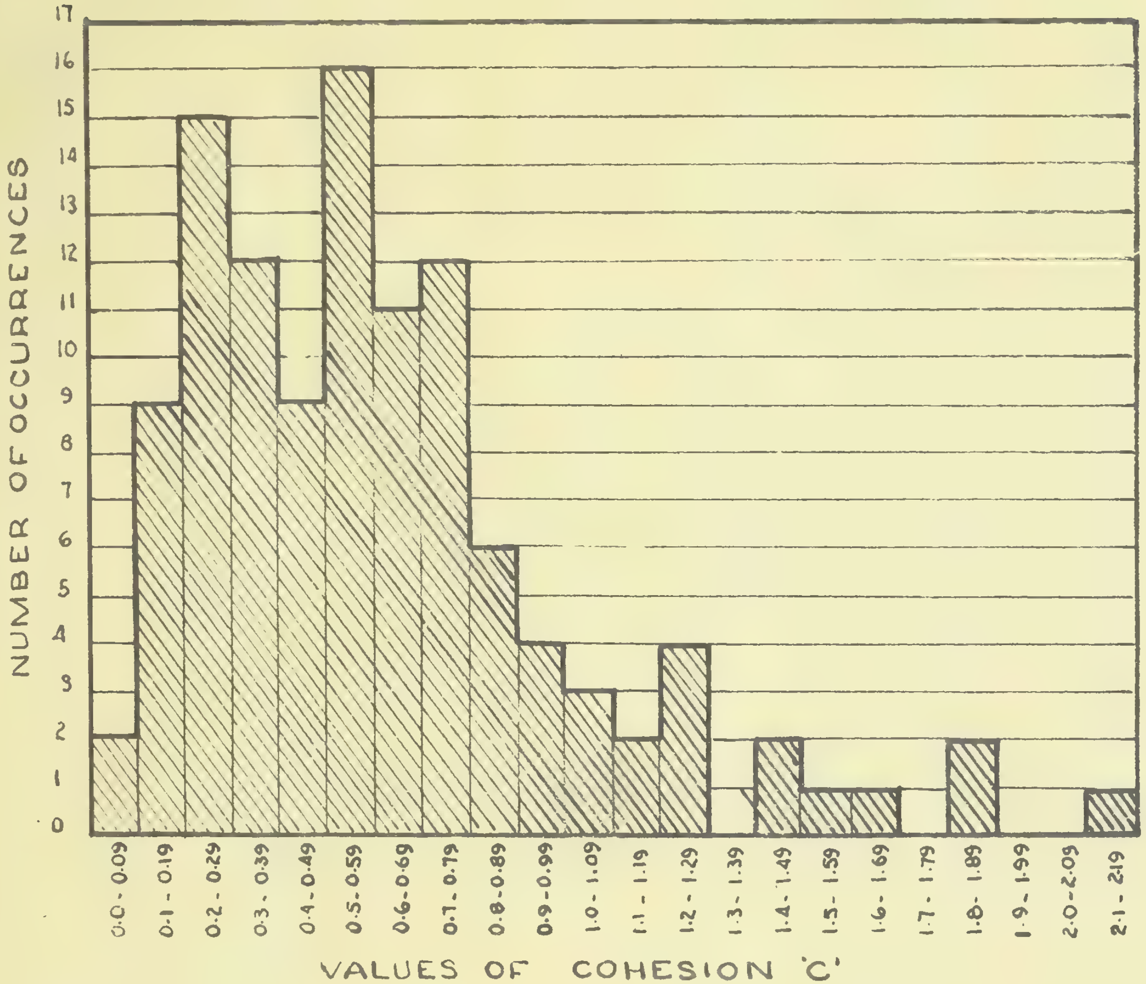
The relationships to be studied are thus reduced to:

1) subgrade support versus angle of internal friction
2) subgrade support versus lateral pressure at the point of inflection

4. Study of Data

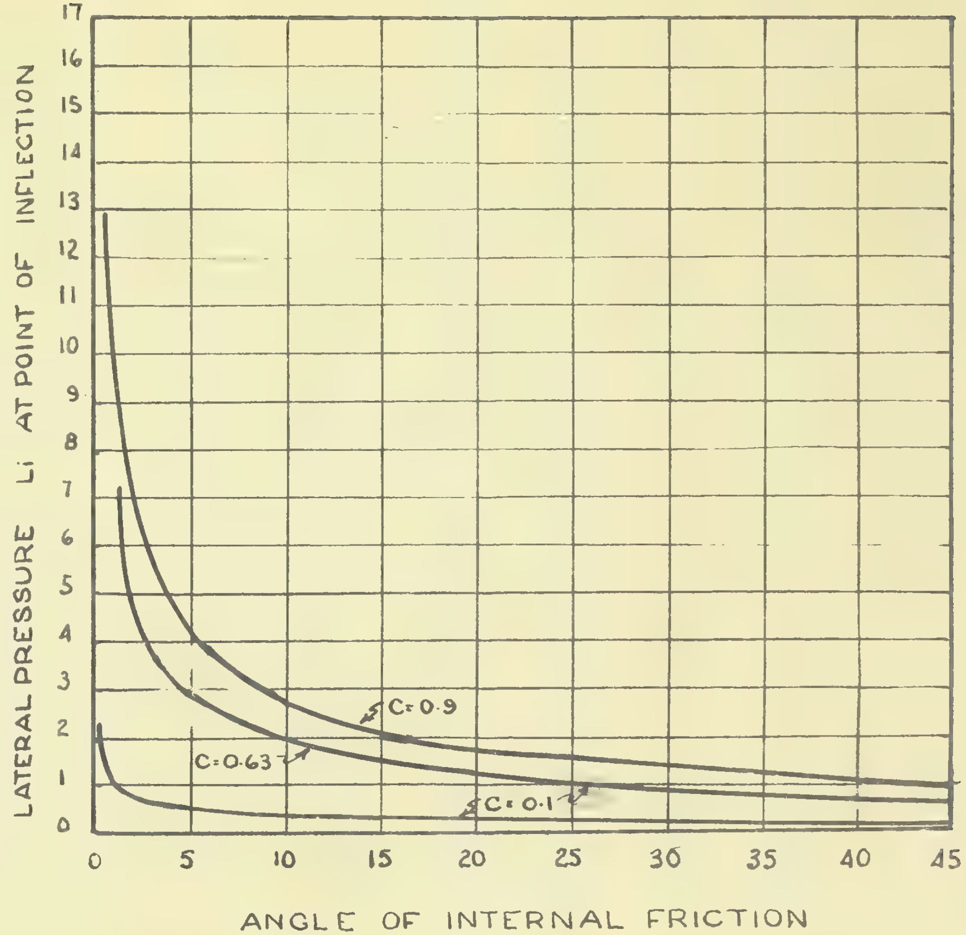
The information that was available was limited to the Lethbridge, Saskatoon, Grande Prairie and Fort St. John airports. Dr. McLeod also includes the Winnipeg and Regina airports. However, they constituted only a small portion of the data and satisfactory conclusions may be obtained without their inclusion. A plot was made of the number of occurrences of values of 'C' between certain ranges (Fig. 7). It was found that 75% of the values of 'C' were between 0.1 and 0.79 tons/sq.ft. and 81% between 0.1

FIGURE 7



NUMBER OF OCCURRENCES OF A GIVEN VALUE OF COHESION D.O.T TRIAXIAL COMPRESSION TEST DATA

FIGURE 8



LATERAL PRESSURE AT POINT OF INFLECTION VS. ANGLE OF INTERNAL FRICTION FOR VARIOUS VALUES OF COHESION FOR TRIAXIAL COMPRESSION TEST

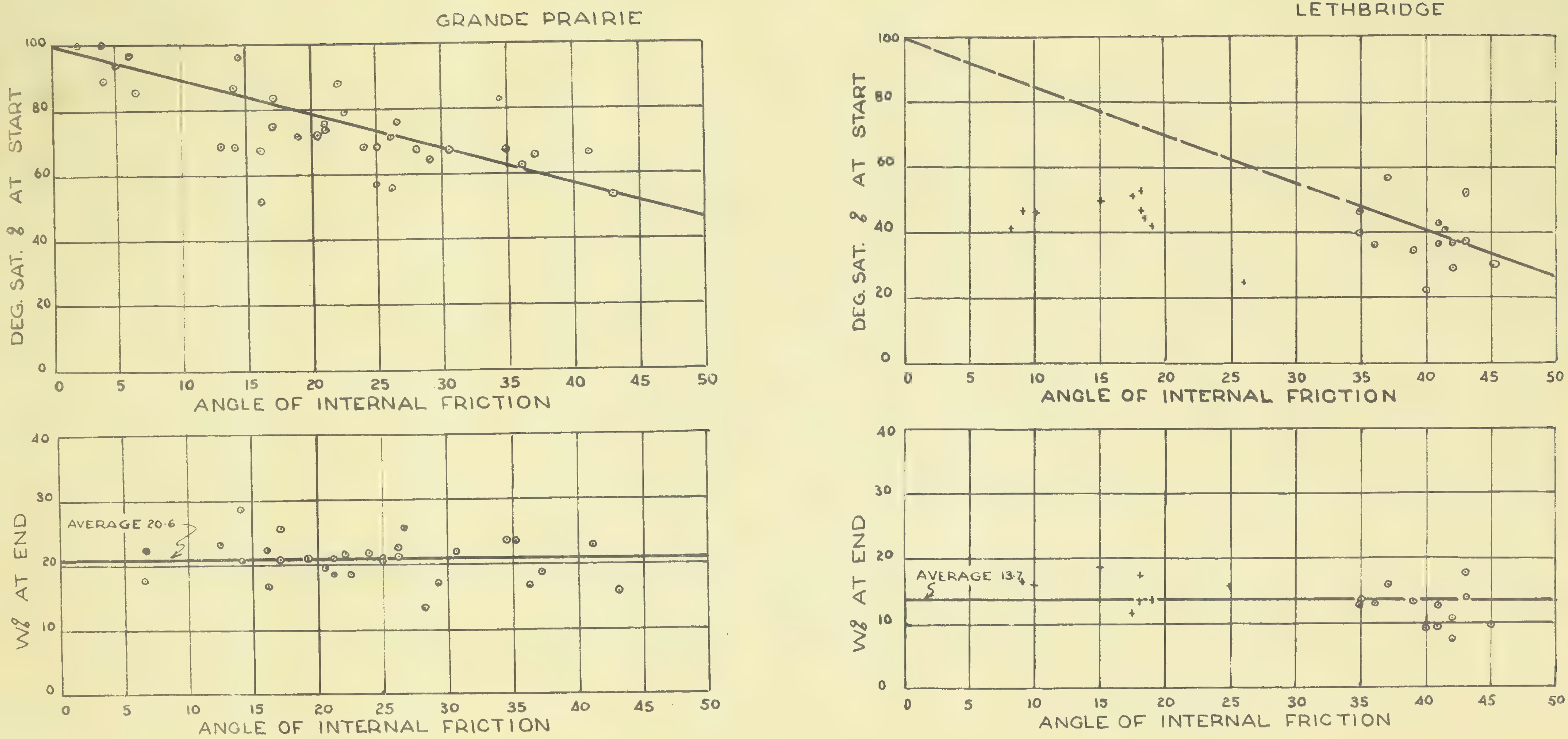


FIG 9, W% AT END OF TEST, DEGREE OF SATURATION AT START VS. ANGLE OF INTERNAL FRICTION
TRIAXIAL COMPRESSION TEST FOR COHESIVE SUBGRADES

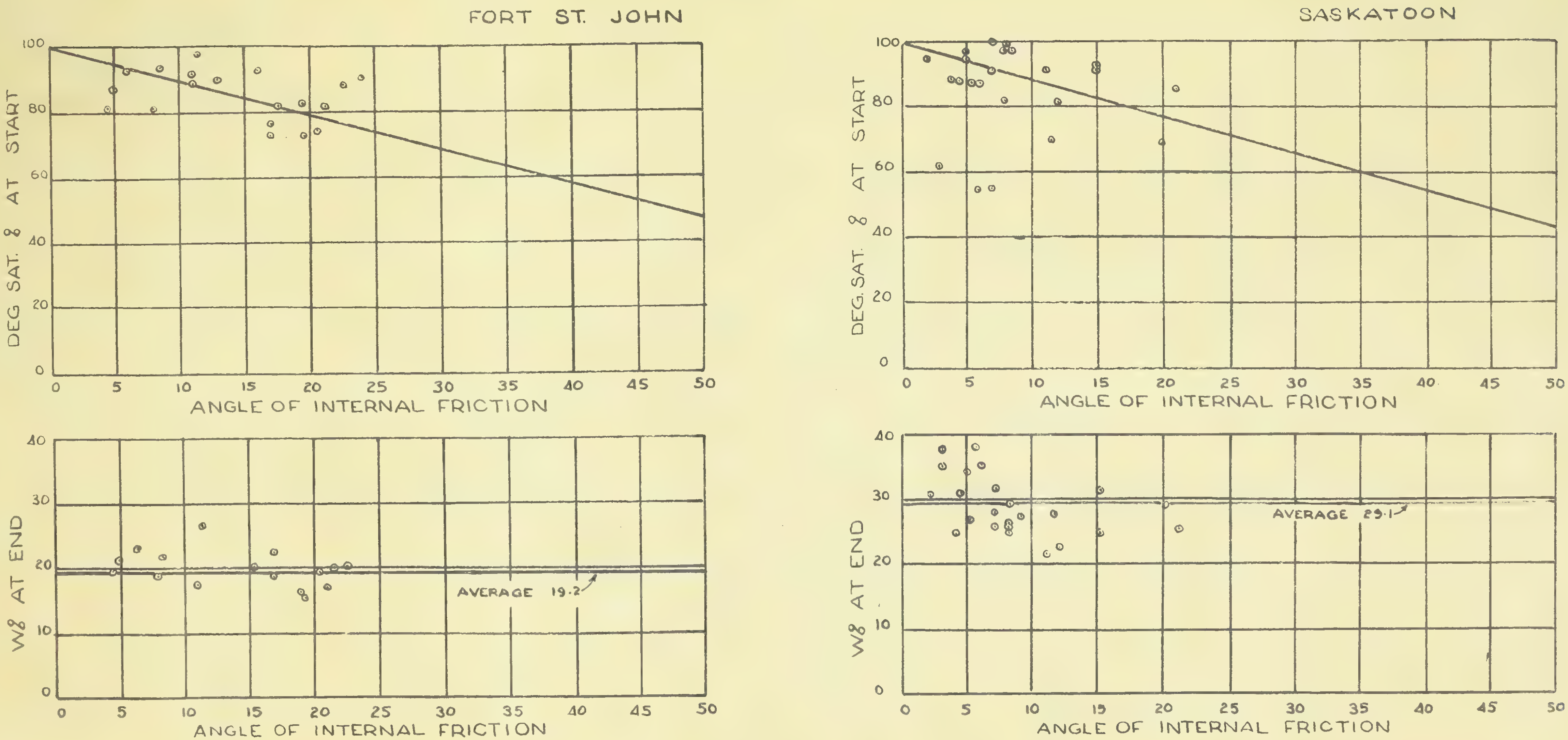
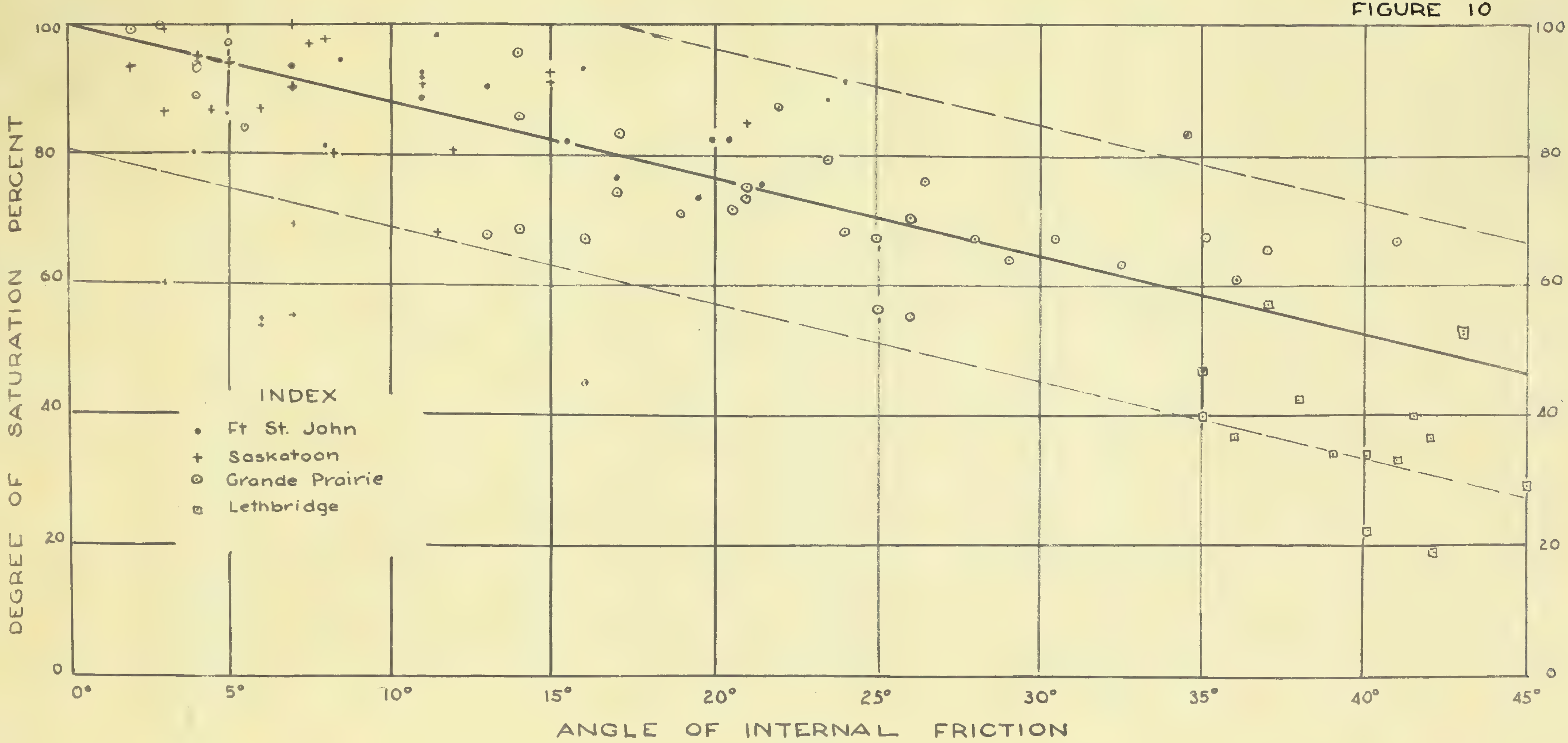


FIG 9, W% AT END OF TEST, DEGREE OF SATURATION AT START VS. ANGLE OF INTERNAL FRICTION
TRIAXIAL COMPRESSION TEST FOR COHESIVE SUBGRADES

FIGURE 10



DEGREE OF SATURATION, PERCENT, VS. ANGLE OF INTERNAL FRICTION
FOR COHESIVE, AIRPORT SUBGRADES

and 0.89 tons/sq.ft. Three curves were then drawn showing the relationship between L_i and ϕ for values of $C = 0.1$, $C = 0.9$ and $C = 0.63$ tons/sq.ft. (Fig. 8).

The curve for $C = 0.63$ gives the average relationship and the other two curves enclose a 'band' from which 81% of the values of L_i may be determined. It will be noted that the band is relatively narrow, even if the remainder of the 'C' values are considered, and thus L_i is primarily a function of 'ϕ' for the values of 'C' that were obtained. A similar treatment may be made of the V_i and 'ϕ' relationship but the band is very wide. Since Dr. McLeod reports no relationship for the latter plot, it indicates that the subgrade support does not depend on 'C' and a relationship was shown between subgrade support and L_i because the effect of 'C' was not appreciable. The relationship that has to be studied is thus reduced to subgrade support versus angle of internal friction.

It was then decided to investigate whether ϕ was an independent variable affecting the subgrade support, or that both ϕ and the subgrade support depended on the moisture contents and degree of saturation. Plots were made of 'ϕ' versus degree of saturation and 'ϕ' versus moisture content. Fig. 9 shows these plots for the individual airports and Fig. 10, 'ϕ' versus degree of saturation for all four airports. The points marked + in Fig. 9 for the Lethbridge airport have been omitted from Fig. 10. The data appears to be in error both in the 'ϕ' versus degree of saturation and the compressive strength versus moisture content in Part II and their omission is justified.

Considerable deviation from the best 'fit line' is noted in Fig. 9 and Fig. 10 but does appear in excess to that shown in Dr. McLeod's graphs. A relationship was indicated between 'ϕ' and the degree of saturation and that 'ϕ' is independent of the moisture content.

Figures 1, 2,3,4,5 and 6 have been reproduced from Dr. McLeod's Report. Figures 7,8,9 and 10 are original in this thesis.

5. Conclusions

a. It has been indicated that subgrade support is influenced, if at all, only to a very slight extent by the cohesion, 'C'.

b. A relationship has been indicated between subgrade support and 'ϕ' and in turn with 'ϕ' and the degree of saturation. It thus follows that both 'ϕ' and the subgrade support are functions of the degree of saturation which is an independent variable. No inconsistency results if the subgrade support is considered dependent on the degree of saturation and 'ϕ' has been used as a 'parameter'. Dr. McLeod's report may be criticized on this basis for not investigating the degree of saturation and moisture contents and their effect on subgrade support.

c. The angle of internal friction has been shown to be independent of the moisture content. The same conclusion is obtained for saturated cohesive soils from the Casagrande two-curve hypothesis.

ACKNOWLEDGEMENT

The writer wishes to express his gratitude to Dean R. M. Hardy of the University of Alberta, Department of Civil Engineering, under whose direction the work was performed.

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